Enclosure 1

Cooper Nuclear Station
Flood Hazard Reevaluation Report

(~~Security-Related Version~~)

~~SECURITY-RELATED INFORMATION - WITHHOLD UNDER 10 CFR 2.390~~

Nebraska Public Power District

Always there when you need us

Cooper Nuclear Station
FLOOD HAZARD REEVALUATION REPORT

SL-012450

Revision 0

Project No.: 11784-017

January 2015

☒ Safety-Related ☐ Non-Safety-Related

Sargent & Lundy LLC

55 East Monroe Street • Chicago, IL 60603 USA • 312-269-2000

www.sargentlundy.com

LEGAL NOTICE

This report was prepared by Sargent & Lundy, L.L.C. ("S&L"), expressly for the sole use of Nebraska Public Power District ("Client") in accordance with the agreement between S&L and Client. This Deliverable was prepared using the degree of skill and care ordinarily exercised by engineers practicing under similar circumstances. Client acknowledges: (1) S&L prepared this Deliverable subject to the particular scope limitations, budgetary and time constraints, and business objectives of the Client; (2) information and data provided by others may not have been independently verified by S&L; and (3) the information and data contained in this Deliverable are time-sensitive and changes in the data, applicable codes, standards, and acceptable engineering practices may invalidate the findings of this Deliverable. Any use or reliance upon this Deliverable by third parties shall be at their sole risk.

SIGNATURES

Preparer: Section 1	T. M. Karas	01-26-15 Date
Reviewer: Section 1	M. Kaiseruddin	1/26/2015 Date
Reviewer: Section 1 (Client comment resolution)	J. W. Dingler	1-26-2015 Date
Preparer: Sections 2 and 3 (except Section 2.3)	M. Salehi	1/26/2015 Date
Reviewer: Sections 2 and 3 (except Section 2.3)	N. M. Patel	1/26/2015 Date
Preparer: Section 2.3	N. M. Patel	1/26/2015 Date
Reviewer: Section 2.3	M. Salehi	1/26/2015 Date
Preparer: Sections 4 and 5	M. Nienaber (NPPD)	1/26/2015 Date
Approver:	B. E. Jelke	1/26/15 Date

TABLE OF CONTENTS

LEGAL NOTICE II

SIGNATURES III

TABLE OF CONTENTS IV

LIST TABLES IX

LIST OF FIGURES XI

LIST OF ACRONYMS AND ABBREVIATIONS XV

INTRODUCTION / PURPOSE XIX

1. SITE INFORMATION RELATED TO THE FLOOD HAZARD 1-1

1.1 Detailed Site Information 1-1

1.1.1 Site Layout 1-1

1.1.2 Spatial Data Sets 1-2

1.1.3 Elevation of Structures, Systems, and Components (SSCs) 1-2

1.1.4 Topography 1-2

1.1.5 Missouri River and Tributaries 1-2

1.2 Current Design Basis Flood Elevations 1-4

1.2.1 Summary of CNS External Flood Design and Licensing Basis 1-4

1.2.2 CLB Local Intense Precipitation (LIP) 1-6

1.2.3 CLB Flooding in Streams and Rivers 1-7

1.2.4 CLB Dam Breaches and Failures 1-7

1.2.5 CLB Storm Surge 1-7

1.2.6 CLB Seiche 1-7

1.2.7 CLB Tsunami 1-7

1.2.8 CLB Ice-Induced Flooding 1-7

1.2.9 CLB Channel Migration or Diversion 1-7

1.2.10 CLB Combined Effects 1-8

1.2.11 CLB Associated Effects 1-8

1.3 Flood-Related Changes and Flood Protection Changes 1-10

1.4 Changes to the Watershed or to the Local Area 1-11

1.5 Current Design Basis Flood Protection and Mitigation Features 1-12

1.6 Additional Site Detail 1-16

1.7 References 1-17

1.8 Tables 1-18

1.9 Figures 1-21

2. FLOOD HAZARD REEVALUATION 2-1

2.1 Local Intense Precipitation (LIP) 2-1

2.1.1 Probable Maximum Precipitation Depths 2-1
2.1.2 Drainage Areas and Local Drainage Parameters 2-2
2.1.3 Peak Discharges 2-3
2.1.4 Hydraulic Model Setup 2-3
2.1.5 Effect of LIP 2-6
2.1.6 References 2-6
2.1.7 Tables 2-8
2.1.8 Figures 2-15

2.2 Flooding in Streams and Rivers (PMF) 2-32

2.2.1 Probable Maximum Precipitation (PMP) 2-33
2.2.2 PMP Runoff Hydrographs 2-35
2.2.3 Water Level Determinations 2-37
2.2.4 Combined Effects 2-44
2.2.5 Associated Flooding Impacts 2-46
2.2.6 References 2-48
2.2.7 Tables 2-51
2.2.8 Figures 2-62

2.3 Dam Breaches and Failures 2-91
2.3.1 Flood Protection Level at the CNS Site 2-93
2.3.2 Hydrologic Evaluation 2-93
2.3.3 Hydraulic Evaluation 2-97
2.3.4 Combined Effects 2-100
2.3.5 Associated Flooding Impacts 2-101
2.3.6 References 2-105
2.3.7 Tables 2-108
2.3.8 Figures 2-127
2.4 Storm Surge 2-154
2.4.1 References 2-154
2.5 Seiche 2-155
2.5.1 References 2-155
2.6 Tsunami 2-156
2.6.1 References 2-156
2.7 Ice-Induced Flooding 2-157
2.7.1 Methodology 2-157
2.7.2 Most Severe Historical Ice Jam Event 2-157
2.7.3 Upstream Breach of an Ice Dam 2-158
2.7.4 Downstream Ice Jam and Resulting Backwater 2-158
2.7.5 Effect of Ice-Induced Flooding 2-158
2.7.6 References 2-159
2.7.7 Figure 2-160
2.8 Channel Migration or Diversion 2-162
2.8.1 Historical Channel Migration or Diversion 2-162
2.8.2 Regional Topographic Evidence 2-163
2.8.3 Ice Causes 2-165
2.8.4 Flooding of Site Due to Channel Migration or Diversion 2-165
2.8.5 Human-Induced Changes of Channel Diversion 2-166

2.8.6 Conclusions 2-166
2.8.7 References 2-167
2.8.8 Figures 2-170

2.9 Combined Effects 2-185
2.9.1 References 2-185

3. COMPARISON OF CURRENT AND REEVALUATED FLOOD-CAUSING MECHANISMS 3-1

3.1 Local Intense Precipitation 3-1

3.2 Flooding in Streams and Rivers 3-1

3.3 Dam Breaches and Failures 3-2

3.4 Storm Surge 3-3

3.5 Seiche 3-3

3.6 Tsunami 3-3

3.7 Ice-Induced Flooding 3-3

3.8 Channel Migration or Diversion 3-4

3.9 Combined Effects 3-4

3.10 Associated Effects 3-4
3.10.1 Hydrostatic and Hydrodynamic Loads 3-4
3.10.2 Debris Loads 3-5
3.10.3 Erosion and Sedimentation 3-5

3.11 Other Pertinent Factors 3-5
3.11.1 Flood Duration 3-5
3.11.2 Overtopping 3-5
3.11.3 Inundation 3-6

3.12 Conclusions 3-6

3.13 References 3-6

3.14 Tables 3-7

4. INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED 4-1

4.1 Regulatory Background 4-1

4.2 Evaluation of the Impact of the Reevaluated Flood Levels on Structures, Systems, and Components (SSCs) 4-1

4.3 Interim Evaluation and Actions Taken or Planned for CNS 4-2

4.3.1 Dam Failure 4-2

4.3.2 Channel Migration or Diversion 4-3

4.4 Reference 4-3

5. ADDITIONAL ACTIONS 5-1

LIST TABLES

Table 1.8-1: Missouri River Dams 1-19

Table 1.8-2: Design Evaluation Summary 1-19

Table 1.8-3: Missouri River Drainage Area 1-20

Table 2.1-1: PMP Values and Intensities at the CNS Site 2-9

Table 2.1-2: Peak Discharge 2-9

Table 2.1-3: Drainage Area and Peak Flow at each Cross Section – Zone A 2-10

Table 2.1-4: Drainage Area and Peak Flow at each Cross Section – Zone B 2-11

Table 2.1-5: Drainage Area and Peak Flow at each Cross Section – Zone C 2-12

Table 2.1-6: Drainage Area and Peak Flow at each Cross Section – Zone D 2-13

Table 2.1-7: PMP Water Levels 2-14

Table 2.2-1: Point 3 (shown in Figure 2.2-4) Adjusted Precipitation Depths Based on HMR 52 2-52

Table 2.2-2: Model Input Parameter Definitions and References 2-53

Table 2.2-3: Summary of Unsteady Computational Parameters 2-55

Table 2.2-4: Location of Inflow Hydrographs 2-55

Table 2.2-5: Comparison of Manning's Roughness Coefficients used in the Model to Standard Values 2-56

Table 2.2-6: NSE Coefficient Summary and Peak Discharge Comparison for 2011 Validation Simulation 2-56

Table 2.2-7: Inflow Hydrograph Location Summary for PMF Simulations 2-57

Table 2.2-8: PMF Simulations Peak Magnitude and Time at RM 556.16 2-59

Table 2.2-9: Monitoring Point Summary 2-59

Table 2.2-10: Calculation of Wind-Driven Waves and Wind Setup Approaching CNS 2-60

Table 2.2-11: Submerged Embankment Results 2-60

Table 2.2-12: Wave Runup Approaching CNS Main Building Complex 2-61

Table 2.2-13: Forces on Intake Structure 2-61

Table 2.2-14: Debris Impact Loads 2-61

Table 2.3-1: Salient Features of Missouri River Mainstem Dams 2-109

Table 2.3-2: Dam Break Parameters for Hypothetical Dams 2-110

Table 2.3-3: HEC-HMS Model Input Parameters and Definitions 2-113

Table 2.3-4: Hypothetical Dam Outflow Summary from HEC-HMS Model 2-115

Table 2.3-5: 500-Year Flow on Missouri River at Drainage Locations 2-118

Table 2.3-6: Peak Flow at HEC-RAS Inflow Locations 2-119

Table 2.3-7: Non-System Dam Failure Peak Stage at CNS and Plant Flood Protection Level 2-121

Table 2.3-8: Peak Stage/Flow at CNS for Combined System and Non-System Dam Failure 2-121

Table 2.3-9: Maximum Average Velocities at CNS - Combined System and Non-System Dam Failure 2-122

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Table 2.3-10: Refined Maximum Velocity at CNS [redacted] Hydrologic and Non-System Dam Failure 2-122

Table 2.3-11: Calculation of Wind-Driven Waves and Wind Setup Approaching CNS 2-123

Table 2.3-12: Summary of Wind Setup, Wave Runup, and Total Water Levels at SSCs 2-123

Table 2.3-13: Summary of Water Depth above Roof at Important Plant Structures 2-124

Table 2.3-14: Summary of Water Depth above Bridge Deck at Important Bridges 2-124

Table 2.3-15: Resultant Forces and Elevations on Main Building Complex Structures 2-125

Table 2.3-16: Resultant Forces and Elevations on Intake Structure 2-125

Table 2.3-17: Debris Impact Loads 2-126

Table 3.14-1: Current Licensing Flood Elevations and Reevaluated Flood-Causing Mechanisms 3-8

Table 3.14-2: Debris Impact Loads Summary 3-9

LIST OF FIGURES

Figure 1.9-1: Site Location 1-22

Figure 1.9-2: Missouri River Tributaries 1-23

Figure 1.9-3: Overhead View of CNS 1 1-24

Figure 1.9-4: Overhead View of CNS 2 1-25

Figure 1.9-5: Missouri River Reservoir System 1-26

Figure 1.9-6: Missouri River Path in 1879 at the Site of CNS 1-27

Figure 1.9-7: Aerial Photograph of the Plant Site in 1971 1-28

Figure 1.9-8: Aerial Photograph of CNS Showing Flooding 1-29

Figure 2.1-1: Location of Security Barriers 2-16

Figure 2.1-2: Drainage Areas for LIP Evaluation 2-17

Figure 2.1-3: Offsite Drainage Area behind USACE Levee 2-18

Figure 2.1-4: Cross Sections for LIP Evaluation - Zone A 2-19

Figure 2.1-5: Cross Sections for LIP Evaluation - Zone B 2-20

Figure 2.1-6: Cross Sections for LIP Evaluation - Zone C 2-21

Figure 2.1-7: Cross Sections for LIP Evaluation - Zone D 2-22

Figure 2.1-8: HEC-RAS Cross Sections for Zone A 2-23

Figure 2.1-9: HEC-RAS Cross Sections for Zone B (sheet 1 of 2) 2-24

Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 1 of 6) 2-26

Figure 2.2-1: Location Map 2-63

Figure 2.2-2: PMP Position 1 - Sheet a (see subsequent Sheets b, c, and d for PMP Positions 2 to 4) 2-64

Figure 2.2-3: Geographic Distribution of Basins of Influence for Missouri River Drainage above CNS 2-68

Figure 2.2-4: PMP Spatial Distribution over Platte River Basin (P3 Centroid; 41.35N, 96.20W) 2-69

Figure 2.2-5: Antecedent Final Model PMF Hydrographs for P1, P2, P3, and P4 2-70

Figure 2.2-6: Subsequent Storm Final Model PMF Hydrographs for P1, P2, P3, and P4 2-71

Figure 2.2-7: Missouri River Junctions 2-72

Figure 2.2-8: Full Floodplain PMF Hydrographs for Gavins Point Release of 160,000 cfs 2-73

Figure 2.2-9: PMF Hydrographs for 2-D Model Upstream Boundary RM 556.16 2-74

Figure 2.2-10: PMF Downstream Boundary Condition Rating Curve RM 510.03 2-75

Figure 2.2-11: Study Area Overview for the 2-D Model 2-76

Figure 2.2-12: Spatial Distribution of Land Use Classes to Determine Manning's Roughness Coefficients 2-77

Figure 2.2-13: Computational Mesh Overview 2-78

Figure 2.2-14: Computational Mesh Near CNS 2-79

Figure 2.2-15: 2-D Model Inflow Hydrographs 2-80

Figure 2.2-16: Comparison of Discharge Hydrographs through the Study Reach 2-81

Figure 2.2-17: Contours of Maximum Water Surface Elevation for PMF Event – Large Scale 2-82

Figure 2.2-18: Contours of Maximum Water Surface Elevation for PMF Event – Small Scale 2-83

Figure 2.2-19: Contours of Maximum Depth for PMF Event - Overview 2-84

Figure 2.2-20: Contours of Maximum Depth for PMF Event – Small Scale 2-85

Figure 2.2-21: Contours of Maximum Velocity for PMF Event – Small Scale 2-86

Figure 2.2-22: Fetches from Embankment Points of Interest 2-87

Figure 2.2-23: Labeled Points of Interest 2-88

Figure 2.2-24: (b)(3):16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F) and Locations of Interior Runup Calculations 2-89

Figure 2.2-25: Expected Maximum Extent of Wave Runup at Representative Locations 2-90

Figure 2.3-1: Non-System Dams Downstream of Gavins Point Dam 2-128

Figure 2.3-2: Inconsequential Non-System Dams Downstream of Gavins Point Dam 2-129

Figure 2.3-3: HEC-HMS Plan for Missouri River below (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam to Omaha, NE 2-130

Figure 2.3-4: HEC-HMS Plan for Missouri River from Omaha, NE to CNS 2-131

Figure 2.3-5: Dam Failure Hydrographs at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam due to System Hydrologic Dam Failure 2-132

Figure 2.3-6: Dam Failure Hydrographs at Dam due to System Sunny-Day Dam Failure 2-133

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Figure 2.3-7: Selected Dam Failure Hydrographs at Dam due to System Dam Failure 2-134

Figure 2.3-8: Estimated Peak Stage and Peak Flow at CNS for Non-System Dam Failure 2-135

Figure 2.3-9: Estimated Peak Stage at CNS due to System and Non-System Dam Failure 2-136

Figure 2.3-10: Estimated Peak Discharge at CNS due to System and Non-System Dam Failure 2-137

Figure 2.3-11: Velocity Distribution Segments at HEC-RAS Cross Section 532.65 (CNS Site) 2-138

Figure 2.3-12: Max. Right Bank Velocity & Total Discharge - Hydrologic Failure (RM 532.65) ..... 2-139 (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-13: Max. Channel Velocity & Total Discharge - (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic Failure (RM 532.65) 2-140

Figure 2.3-14: Max. Left Bank Velocity & Total Discharge Hydrologic Failure (RM 532.65) 2-141

Figure 2.3-15: WSEL with Fetch Overlay 2-142

Figure 2.3-16: Fetch Directions for Waves Approaching Plant East and Plant West 2-143

Figure 2.3-17: Fetch Directions for Waves Approaching Plant North and Plant South 2-144

Figure 2.3-18: Plan View of Pressure Calculation Surfaces for (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) 2-145

Figure 2.3-19: Cross Sections and Channel Bank Locations at CNS RMs 532.65, 532.53, and 532.49 2-146

Figure 2.3-20: Max. Right Bank Velocity & Total Discharge (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic Failure (RM 532.49) 2-147

Figure 2.3-21: Water Surface Elevation & Total Discharge - Hydrologic Failure (RM 532.65) 2-148

Figure 2.3-22: Max. Right Bank Bed Shear & Total Discharge - Hydrologic Failure (RM 532.49) ..... 2-149 (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-23: Inundation Map for (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic Dam Failure Scenario (Large-Scale Map) 2-150

Figure 2.3-24: Inundation Map for Hydrologic Dam Failure Scenario (Medium-Scale Map) 2-151

Figure 2.3-25: Inundation Map for Hydrologic Dam Failure Scenario (Small-Scale Map) 2-152

Figure 2.3-26: Bridge Location Map 2-153

Figure 2.7-1: Hydrologic Unit Code (HUC) for Different Watersheds Upstream and Downstream of the Site .. 2-161

Figure 2.8-1: 1879 Land Cover 2-171

Figure 2.8-2: 1940 Aerial Photography 2-172

Figure 2.8-3: 1965 Aerial Photography 2-173

Figure 2.8-4: 1993 Aerial Photography 2-174

Figure 2.8-5: 1999 Aerial Photography 2-175

Figure 2.8-6: 2003 Aerial Photography 2-176

Figure 2.8-7: 2005 Aerial Photography 2-177

Figure 2.8-8: 2010 Aerial Photography 2-178

Figure 2.8-9: Hydric Soils 2-179

Figure 2.8-10: Geomorphic Soil Description 2-180

Figure 2.8-11: Soil Textures 2-181

Figure 2.8-12: 2011 Landsat Thematic Mapper Surface Reflectance On-demand 2-182

Figure 2.8-13: 2012 Aerial Photography 2-183

Figure 2.8-14: 2011 Flood Breach Locations 2-184

LIST OF ACRONYMS AND ABBREVIATIONS

Acronym or Abbreviation	Explanation
1-, 2-, 3-D	one-dimensional, two-dimensional, three-dimensional
ac	acre
ac-ft	acre-feet
ACES	Automated Coastal Engineering System
AEC	U.S. Atomic Energy Commission
AEP	annual exceedance probability
ANS	American Nuclear Society
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
BSNP	bank stabilization and navigation project
BWR	boiling water reactor
CFR	Code of Federal Regulations
CEDAS	Coastal Engineering Design & Analysis System
CEM	Coastal Engineering Manual
cfs	cubic feet per second
CLB	Current Licensing Basis
CNS	Cooper Nuclear Station
COL	combined operating license
CRREL	Cold Regions Research and Engineering Laboratory
DEM	digital elevation model
ERP	Elevated Release Point
ESP	early site permit
ESPA	early site permit application
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
FHR	flood hazard reevaluation

Acronym or Abbreviation	Explanation
FIS	flood insurance study
ft	feet
fps	feet per second
FSAR	Final Safety Analysis Report
GI	Generic Issue
GIS	geographical information system
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical Hazard Assessment
H_{m0}	spectral significant wave height
HMR	Hydrometeorological Report
hr	hour
HTab	Hydraulic Table
HU	hydrologic unit
HWM	high-water mark
in	inch
in/hr	inch per hour
IPEEE	Individual Plant Examination for External Events
ISFSI	Independent Spent Fuel Storage Installation
ISG	Interim Staff Guidance
LiDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
LOB	Left Overbank
m	meter
mi	mile
min	minute
mph	mile per hour
MPF	Multi-Purpose Facility

Acronym or Abbreviation	Explanation
MSL	mean sea level
MWe	megawatt electric
MWt	megawatt thermal
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NCHRP	National Cooperative Highway Research Program
NDNR	Nebraska Department of Natural Resources
NEmbE	north embankment east
NGVD29	National Geodetic Vertical Datum of 1929
NID	National Inventory of Dams
NLSWE	Non-linear Shallow Water Equation
NOAA	National Oceanic and Atmospheric Administration
NPPD	Nebraska Public Power District
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NSE	Nash-Sutcliffe efficiency coefficient
NTTF	Near-Term Task Force
NWS	National Weather Service
PA	Protected Area
PBL	planetary boundary layer
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PMT	Probable Maximum Tsunami
psf	pound per square foot
RCIC	Reactor Core Isolation Cooling
RHR	Residual Heat Removal
RG	Regulatory Guide
RM	River Mile

Acronym or Abbreviation	Explanation
ROB	Right Overbank
SCS	Soil Conservation Services
SER	Safety Evaluation Report
SGTS	Standby Gas Treatment System
SSC	structure, system, and component
T_p	peak wave period
tpd	tons per day
UMRSFFS	Upper Mississippi River System Flow Frequency Study
USACE	U.S. Army Corps of Engineers
USACE-OD	USACE Omaha District
USAR	Updated Safety Analysis Report
USBR	United States Bureau of Reclamation
USCG	United States Coast Guard
USDA	United States Department of Agriculture
USGS	United States Geological Survey
VBS	vehicle barrier system
WEmbN	west embankment north
WEmbS	west embankment south
WEmbW	west embankment west
WRF	width reduction factor
WSEL	water surface elevation

INTRODUCTION / PURPOSE

Following the accident at the Fukushima Dai-ichi nuclear power plant resulting from the March 11, 2011, Great Tōhoku Earthquake and subsequent tsunami, the U.S. Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF). The NTTF Charter tasked the NTTF with conducting a systematic and methodical review of NRC processes and regulations to determine if the agency should make additional improvements to its regulatory system. Ultimately, a comprehensive set of recommendations was developed.

In response to the NTTF recommendations and pursuant to Sections 161.c, 103.b, and 182.a of the Atomic Energy Act of 1954, as amended, and Title 10 of the Code of Federal Regulations (10 CFR), Section 50.54(f), the NRC has requested information from all operating power plant licensees. The purpose of the request is to gather information to:

- Reevaluate seismic and flooding hazards at U.S. operating reactor sites.
- Facilitate the NRC's determination if there is a need to update the design basis and structures, systems, and components (SSCs) important to safety to protect against the updated hazards at operating reactor sites.
- Address Generic Issue (GI) 204 regarding flooding of nuclear power plant sites following upstream dam failures.

The information request relating to flooding hazards requires licensees to reevaluate their sites applying present-day regulatory guidance and methodologies being used for early site permit (ESP) and combined operating license (COL) reviews, including current techniques, software, and methods used in present-day standard engineering practice to perform the flood hazard studies. The results are compared against the site's current licensing basis (CLB) for protection and mitigation from external flood events.

This report describes the flooding reevaluation performed for Cooper Nuclear Station (CNS). This report satisfies the information requested by Enclosure 2 (Recommendation 2.1: Flooding) of U.S. NRC letter, *Request For Information Pursuant To Title 10 Of The Code Of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, And 9.3, Of The Near-Term Task Force Review Of Insights From The Fukushima Dai-Ichi Accident*, dated March 12, 2012 (Reference 1.7-1).

1. SITE INFORMATION RELATED TO THE FLOOD HAZARD

1.1 DETAILED SITE INFORMATION

Cooper Nuclear Station (CNS) is located on the Nebraska portion of a 1,090-acre (ac) site (Reference 1.7-2, Section 1.1) in Nemaha County, Nebraska and Atchison County, Missouri.

The plant, located approximately 55 miles (mi) south-southeast of Omaha, Nebraska and 100 mi north-northwest of Kansas City, Kansas, is on the west bank of the Missouri River at River Mile (RM) 532.5 located midway between the villages of Brownville and Nemaha, Nebraska. ArcMap was used to show the location of the plant (see Figure 1.9-1). This portion of the Missouri River is referred to as the Lower Brownville Bend. The U.S. Army Corps of Engineers (USACE) has stabilized the channel near the plant using pile dikes and bank protection. This control prevents meandering of the river within the alluvial flood plain (Reference 1.7-3, Section 4.1).

The Missouri River flows past the site in a southeasterly direction under open-river conditions, there being no downstream dams to control the stage of the river. In the vicinity of CNS, levees with crest elevations of approximately 902 feet (ft) mean sea level (MSL) control the river section during flood stages. River flow is controlled by a series of flood control dams on the upper Missouri River, which are operated by the USACE for a number of authorized purposes, including flood control, navigation, irrigation, power, water supply, water quality control, recreation, and fish and wildlife. The Platte River (see Figure 1.9-2) is a major source of flow and joins the Missouri River south of Omaha, Nebraska. Maximum water surface elevation (WSEL) at the CNS site, as reported by the USACE, was 899 ft MSL in 1952 prior to the installation of the upstream control dams. Peak flood flow in 1952 was 414,000 cubic feet per second (cfs), which would have been reduced to approximately 100,000 cfs if today's controls had been in place at that time.

The site coordinates are approximately 40° 21' north latitude and 95° 38' west longitude (Reference 1.7-4, Chapter II, Section 2.1). See Figure 1.9-1, Figure 1.9-3, and Figure 1.9-4 for various views of the plant.

1.1.1 Site Layout

CNS consists of a single boiling water reactor (BWR) with a capacity of 836 megawatt electric (MWe) / 2,419 megawatt thermal (MWt) and first went into service on July 1, 1974 (Reference 1.7-4, Chapter I, Section 1.0).

The principal structures of the station consist of the Reactor Building, Turbine Building (including service area appendages), Control Building, Radwaste Building, Augmented Radwaste Building, Intake Structure, Diesel Generator Building, Drywell and Suppression Chamber, Miscellaneous Circulating Water System Structures (e.g., circulating water conduits, seal well, etc.). (See Reference 1.7-4, Section XII.) Aside from the Intake Structure, the nearest building to the river channel edge is approximately 200 ft away (Reference 1.7-5, Section 2.1).

1.1.2 Spatial Data Sets

Elevations in Section 1 are referenced to MSL for consistency with the CNS Updated Safety Analysis Report (USAR). In this context, MSL is equated to Plant Datum and is distinct from other geodetic vertical datums. Reference 1.7-3 notes that Plant Datum is the same datum used in the USAR.

1.1.3 Elevation of Structures, Systems, and Components (SSCs)

The station site grade level of 903 ft MSL has been raised 13 ft above the natural grade level of 890 ft MSL, in order to bring final grade 1 ft above the existing 902 ft MSL levee constructed by the USACE (Reference 1.7-4, Chapter II, Section 4.2.2.2).

The levee is situated at a nominal elevation of 902 ft MSL. The finished floor elevation is given as 903.5 ft MSL for all Class I structures (Reactor Building, Turbine Building, Diesel Generator Building, Control Building, Radwaste Building, Intake Structure and Controlled Corridor). The Z-Sump is at 890 ft MSL, and the Elevated Release Point (ERP) tower foundations are at 891 ft MSL (Reference 1.7-3, Section 4.0).

1.1.4 Topography

Prior to the construction of CNS, the site was nearly flat with only one or two feet of relief over the majority of the area. General grade elevation was approximately 890 ft MSL. A bluff, approximately 150 ft above the floodplain, running north-south, is located about one mile west of the river near the plant area. An earthen levee parallels the west (right) river bank at a distance of approximately 500 ft. Surface drainage of the site was poor because of the lack of natural relief. The levee also inhibits surface runoff from the area behind the levee to the river (Reference 1.7-6, Section 2.2).

Federal Levee R548 is adjacent to the CNS main plant area. The top elevation of the levee near the site is approximately 902 ft MSL, 1 ft below the nominal plant grade of 903 ft MSL. The levee extends north of the plant, where it terminates at the higher ground near the bluffs. South of CNS, the levee follows the right bank of the Missouri River, curving around to the west and running parallel to the left bank of the Nemaha River, which is a tributary to the Missouri River, and turning north where it meets the higher ground near the town of Nemaha. The top elevation of the levee varies from approximately 902 ft MSL near CNS to approximately 897 ft MSL on the southern end, with top elevations of the levee as low as 894 ft MSL locally. The area enclosed by the levee and the higher bluffs to the west is 4000 ac (Reference 1.7-7, Section 5.1.8).

1.1.5 Missouri River and Tributaries

The Missouri River Basin drains 529,000 square miles (mi^2), including about 9,700 mi^2 located in Canada. The Basin spans 10 states, including all of Nebraska, most of Montana, Wyoming, North Dakota, and South Dakota; about half of Kansas and Missouri; and smaller parts of Iowa, Colorado, and Minnesota. The Missouri River is fed from several major tributaries, including the Yellowstone, Platte, Kansas, Grand, and Osage Rivers. The Missouri River extends 2,619 mi from its uppermost source at Browers Spring on Hell Roaring Creek and 2,321 mi from Three Forks, Montana, where the

Jefferson, Madison, and Gallatin Rivers converge. The Missouri River is the longest river in the United States, draining one-sixth of the country. The Missouri River Mainstem System (System) consists of six dams and reservoirs on the Missouri River located in Montana, North Dakota, South Dakota, and Nebraska. The six System dams are Fort Peck Dam in Montana; Garrison Dam in North Dakota; Oahe Dam, Big Bend Dam, and Fort Randall Dam in South Dakota; and Gavins Point Dam in South Dakota and Nebraska. The System has a capacity to store 73.1 million acre-feet (ac-ft) of water, which makes it the largest reservoir system in North America. Runoff from upstream of the System is stored in the six reservoirs. Water is released from the System as needed for downstream flow support. Released water from the most downstream dam in the System, Gavins Point Dam, flows down the Missouri River, which includes the Bank Stabilization and Navigation Project (BSNP) from Sioux City, Iowa to the mouth near St. Louis, Missouri (Reference 1.7-8 and Figure 1.9-5).

The location of six System dams is noted in Table 1.8-1. There are no dams or similar structures on the Missouri River downstream of the plant site. (See Figure 1.9-2 and Figure 1.9-5 and Reference 1.7-4, Chapter II, Section 4.2.2.1.)

1.2 CURRENT DESIGN BASIS FLOOD ELEVATIONS

This section summarizes key flood and plant information found in the CNS USAR.

1.2.1 Summary of CNS External Flood Design and Licensing Basis

The safety equipment needed to maintain a safe shutdown includes the Diesel Generators, which are near plant grade (903.5 ft MSL), Reactor Core Isolation Cooling (RCIC) and Residual Heat Removal (RHR) pumps located in the Reactor Building basement (859.7 ft MSL); RHR service water booster pumps located in the Control Building basement (882.5 ft MSL); various electrical equipment and switchgear needed to operate decay heat removal systems at elevations ranging from 903.5 to 905.0 ft MSL; and the service water pump motors located at 907.5 ft MSL in the Intake Structure (Reference 1.7-2, Section 2.4.3). The protection of building openings to 906.0 ft MSL, 2.5 ft above the finished floor elevation, is accomplished by deploying temporary flood control barriers. These barriers are deployed at critical grade level openings around and within the Main Building Complex. Engineered flood barriers are used; however, sandbags are available for contingencies, as needed, to supplement the engineered barriers. The material and equipment necessary to implement these protective measures are available at CNS and are inventoried on an annual basis. Special flood fighting equipment includes two portable gasoline-powered pumps and 100 ft minimum (per pump) of non-collapsible hose (Reference 1.7-4, Chapter II, Section 4.2.2.2).

See Table 1.8-2 for a summary of the design evaluation.

1.2.1.1 Flood Levels (USAR)

Information on riverine flooding in the CNS USAR is based on independent studies performed by the USACE and by the U.S. Atomic Energy Commission (AEC). Information from the USACE study is summarized in the USAR and its predecessor, the Final Safety Analysis Report (FSAR). The AEC study is summarized in the CNS Safety Evaluation Report (SER). These studies are discussed in subsequent sections of this report.

The projected upper limit of elevation for a 1,000-year flood discharge is 900 ft MSL and for a 10,000-year flood discharge is 902 ft MSL. The Probable Maximum Flood (PMF) is established at 903 ft MSL with a projected return frequency estimated to be in excess of 1,000,000 years. The PMF is derived by centering a probable maximum rainstorm critically over the drainage area above Brownville. The USACE has estimated that the peak discharge for the PMF is 600,000 cfs (Reference 1.7-4, Chapter II, Section 4.2.2.1). This estimate is based on a qualitative judgment that the peak of a PMF event should be between two to three times the peak of a standard project flood. Preliminary estimates of standard project conditions on the Missouri and Platte Rivers were used in estimating the PMF peak of 600,000 cfs at CNS (Reference 1.7-4, Chapter II, Section 4.2.2.1). Table 1.8-3 summarizes the approximate drainage area (mi^2) for locations of interest along the Missori River.

The effects of a sustained wind in conjunction with a PMF were evaluated by USACE using the procedures described in the U.S. Army Coastal Engineering Research Technical Report 4. Assuming a

45-mile per hour (mph) overwater wind speed and a 903 ft MSL PMF WSEL, the significant shallow-water wave height was computed to be 4.0 ft. The 1% wave height was computed to be 6.7 ft. The runup was computed to be 6.7 ft on a smooth vertical wall located in shallow water and 4.8 ft on a one-on-three riprapped slope. The fetches are of sufficient length that the above waves could be generated with winds from the north through east through south. It should be noted, however, that these wave height values are based on relationships derived from studies made on lakes and oceans, and it is believed that the same parameters would not produce waves as high as those computed above in highly turbulent, fast-flowing river flood waters (Reference 1.7-4, Chapter II, Section 4.2.2.1).

The wave action, as mentioned above, will not affect the plant proper since the nearest building is located about 200 ft from the river edge and is surrounded by grade to elevation 903 ft MSL. Wave energy would be dissipated before reaching any of the main buildings. Wave action at the Intake Structure will not affect the safe shutdown of the plant since the service water pumps and controls are protected by massive reinforced concrete walls and slab up to elevation 919 ft MSL (Reference 1.7-4, Chapter II, Section 4.2.2.2).

Flooding caused by ice blockage is considered credible only at river levels significantly lower than the PMF. Flooding caused by ice blockage would cause water surface elevations below those of the PMF. (Reference 1.7-4, Chapter II, Section 4.2.2.1.)

The failure of a large upstream dam is not considered probable. These dams are massive earthen structures with impervious core walls, large freeboard for wave action, and adequate spillways. The dams are located in a zone of low seismic activity. The Gavins Point Dam, closest dam to the site, is over 275 mi upstream (Reference 1.7-4, Chapter II, Section 4.2.2.1).

These dams are under constant inspection by local USACE personnel and are inspected once a year by a team of professionals. If seismic failure occurred in spite of these circumstances, the failure would most probably be caused by other than instantaneous failure of a major portion of the dam. Under these circumstances, overtopping of Gavins Point Dam, although conceivable, is not likely to occur (Reference 1.7-4, Chapter II, Section 4.2.2.1).

1.2.1.2 USACE Study

The USACE estimated a PMF elevation between 902 ft MSL and 903 ft MSL at CNS. This is based on a Probable Maximum Precipitation (PMP) event occurring upstream of Brownville, Nebraska and a resulting flow rate in the Missouri River of 600,000 cfs. The rainfall event producing this flow rate is described in FSAR Question 2.1 as originating over the Platte River Basin (Reference 1.7-9).

1.2.1.3 AEC Study Safety Evaluation Report

The AEC performed an independent PMF study for CNS, which is summarized in the CNS SER. Based on detailed PMF estimates in the general region of the site, AEC staff conservatively estimated the peak PMF discharge to be about 1,000,000 cfs. This estimate was conservatively considered to reflect the total runoff potential represented by the large drainage area downstream of the USACE dams, plus the relatively minor component that could be contributed by releases from these dams. AEC staff

concluded that the dams should remain intact during a PMF. Additionally, AEC staff constructed an analytical water surface profile model of the Missouri River from below CNS to the Brownville Bridge a few miles upstream and estimated the PMF still WSEL at the site would be 901.2 ft MSL. The model was verified by reproducing historical flood high-water marks using estimates of floodplain geometry and flood runoff rates. Coincident wave action induced by an assumed 45-mph wind was estimated to produce a maximum WSEL of 909.2 ft MSL at the Intake Structure and 905 ft MSL on the other exposed safety-related structures. AEC staff did not consider seismically induced dam failure coincident with PMF credible but considered the effects of a seismically induced failure of one of the largest upstream dams concurrent with flood levels of one half of the PMF and concluded that the water level that could result at CNS is less than the AEC staff's estimated PMF (Reference 1.7-2, Section 2.4.3). This information has been incorporated into the USAR. (See Section 1.2.1.1 of this report.)

1.2.1.4 Conclusions

The 906 ft MSL level for which the station committed to provide protection bounds both the AEC and USACE studies. Therefore, 906 ft MSL should be considered the Current Licensing Basis (CLB) still WSEL due to Missouri River flooding. The wave impact design basis outside of the intake structure is 909.2 ft MSL (Reference 1.7-4, Chapter II, Section 4.2.2.2).

The immediate station area grade is at elevation 903 ft MSL. This is 4 ft above the flood of record, 1 ft above the existing levee, and equal to the highest projected level of the maximum probable natural flood. Overtopping of the levee would occur before flooding of the station would occur (Reference 1.7-4, Chapter II, Section 4.6).

Failure of an upstream dam is considered improbable. However, in the unlikely event that such a failure does occur, it would be approximately three days before the water would reach the site, which is sufficient time to provide protection or to conduct a safe and orderly shutdown of the plant. High water resulting from failure of an upstream dam would overtop the levee before flooding the station. (Reference 1.7-4, Chapter II, Section 4.6).

1.2.2 CLB Local Intense Precipitation (LIP)

From Hydrometeorological Report No. 33 (HMR-33) dated April, 1956 by the U.S. Department of Commerce – Weather Bureau and the USACE (Reference 1.7-10), it was determined that the PMP for the CNS site area is 23.5 inches (in) total rainfall for a 24-hour period. This bounding PMP value was obtained from HMR-33 Figure 17 (August). (See Reference 1.7-4, Chapter II, Section 3.1.3). An independent evaluation by the AEC determined that the PMP 1-hour rainfall rate was 9.7 in per hour (Reference 1.7-2, Section 3.4).

Class I and Class II buildings are protected from the effects of precipitation through the use of roof drains and overflow scuppers. The Reactor Building, Diesel Generator Building, and Control Building use 4-in roof drains and 6-in scuppers. The remaining local site drainage is designed such that any excess rainfall not immediately absorbed into the ground will flow away from the buildings to be discharged into drywells or low lying areas adjacent to the plant site. Therefore, this configuration can safely remove the accumulated water from the PMP rate described in the USAR, Chapter II,

Section 3.1.3, and can also accommodate the 9.7 in per hour (Reference 1.7-11 and Reference 1.7-4, Chapter II, Section 3.2.3) rate estimated by the AEC without adverse effects on the safety-related systems necessary for safe shutdown (Reference 1.7-4, Chapter II, Section 3.2.3).

1.2.3 CLB Flooding in Streams and Rivers

Refer to Section 1.2.1.

1.2.4 CLB Dam Breaches and Failures

The USAR (Reference 1.7-4, Chapter II, Section 4.2.2.1) notes that a dam failure was not considered a credible event. The maximum expected still water level at the site associated with a postulated dam failure was 906 ft MSL, with a minimum warning time of three days (Reference 1.7-4, Chapter II, Section 4.2.2.2). No associated effects (wind-wave, debris, etc.) were considered in the dam failure scenario.

1.2.5 CLB Storm Surge

The CLB is silent on flooding effects from storm surge. However, due to its location, the storm surge flooding mechanism can be screened out for CNS.

1.2.6 CLB Seiche

The CLB is silent on flooding effects from seiche. However, due to its location, the seiche flooding mechanism can be screened out for CNS.

1.2.7 CLB Tsunami

The CLB is silent on flooding effects from tsunami. However, due to its location, the tsunami flooding mechanism can be screened out for CNS.

1.2.8 CLB Ice-Induced Flooding

Flooding caused by ice blockage is considered credible only at river levels significantly lower than the PMF. Flooding caused by ice blockage would produce WSELs below those of the PMF (Reference 1.7-4, Chapter II, Section 4.2.2.1). Thus, the CLB does not include quantitative parameters for ice-induced flooding, but the ice-induced flooding effects are bounded by the PMF.

1.2.9 CLB Channel Migration or Diversion

The CLB discusses this mechanism only with respect to channel migration away from the site due to levee failure on the opposite river bank and the potential loss of the heat sink. Channel migration toward the site was not considered credible.

Changes in river alignment have occurred in the past as a result of meteorological events. The river channel at present is stabilized as a result of permanent revetments constructed under the supervision

of the USACE over a number of years; thereby, making it extremely unlikely that the river would be diverted at Brownville Bend. If, however, in spite of these controls, the river should be diverted, it would most likely occur as a result of the failure of a major upstream dam - either [redacted] Dam or [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b) Dam, although dam failure is not considered credible in the CLB. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7) (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) from the plant site and there would be a three-day warning before the river reached a peak flood stage at CNS. While the river is at peak flood stage at the site, the postulated diversion of the Missouri River might occur upstream. During peak flood stage, reactor shutdown procedures could begin and continue as the flood water slowly receded. With the river diverted and the flood waters fully receded, what might remain in front of the Intake Structure would be a large body of still water cut off from the upstream flow and possibly cut off at a point downstream. After the river diversion occurs, the resulting isolated body of water in the vicinity of the site, fed by groundwater inflow, would retain essentially the same stage characteristics that apply to the current open river as long as the main channel were retained in the existing valley. It was concluded that there would be an adequate supply of water to affect a safe shutdown of the plant (Reference 1.7-4, Chapter II, Sections 4.2.3.1 and 4.2.3.2). The AEC concurred with these findings in the CNS SER, which states that it was considered unlikely that major channel diversions would occur (Reference 1.7-2, Section 2.4.5).

1.2.10 CLB Combined Effects

Combined effects of different flood-causing mechanisms are discussed in Section 1.2.1 through 1.2.8, where applicable.

1.2.11 CLB Associated Effects

Section 9 of Reference 1.7-12 defines "Flood height and associated effects" as:

The maximum still WSEL plus the following factors:

- Wind waves and runup effects.
- Hydrodynamic loading, including debris.
- Effects caused by sediment deposition and erosion.
- Concurrent site conditions, including adverse weather conditions.
- Groundwater ingress.
- Other pertinent factors

1.2.11.1 Wind Waves and Runup Effects

The CLB values for wind waves and runup is 909.2 ft MSL at the Intake Structure and 905 ft MSL for other exposed safety-related structures (Reference 1.7-4, Chapter II, Section 4.2.2.2).

1.2.11.2 Hydrostatic and Hydrodynamic Loads

The principal structures are designed to withstand hydrostatic loads up to the flood protection level of 906 ft MSL with margin (Reference 1.7-4, Chapter II, Section 4.2.2.2).

The CLB is silent on hydrodynamic loads.

1.2.11.3 Debris Loads

The CLB is silent on flooding debris loads (Reference 1.7-4); however, the CLB does discuss a runaway barge (Reference 1.7-4, Chapter II, Section 4.3).

In summary, the Intake Structure and Class I equipment located inside are qualified to withstand a barge impact. Additionally, a barge that impacts the Intake Structure guide wall will, therefore, have little if any effect on plant operations.

1.2.11.4 Erosion and Sedimentation

Effects of erosion and sedimentation during extreme flooding events are not analyzed in the CLB.

1.2.11.5 Concurrent Site Conditions

The CNS CLB (Reference 1.7-4) does not specifically discuss evaluation of concurrent site conditions.

1.2.11.6 Groundwater Ingress

The CNS CLB does not specifically address or quantify groundwater in-leakage from the site subsurface water table. Groundwater level is monitored/logged once per shift based on level indicators located within site well(s) providing plant water supply. According to CNS personnel, site groundwater level normally fluctuates between 875 ft MSL to 885 ft MSL with the highest recorded level of 900.8 ft MSL during the May 2011 Missouri River flood event (Reference 1.7-5, Section 3.7).

1.2.11.7 Other Pertinent Factors

The Maintenance Procedure directs the CNS maintenance personnel in the installation of temporary flood control barriers and features. The procedure provides a system of primary barriers at the openings of the outside walls of the Main Building Complex to seal the buildings up to elevation 906 ft MSL. Secondary barriers are provided strategically inside the buildings to control any minor leakage past the primary barriers and to segregate the building complex into small areas that can be more easily protected (Reference 1.7-5, Section 3.5.)

1.3 FLOOD-RELATED CHANGES AND FLOOD PROTECTION CHANGES

The plant design features and their functional requirements that provide protection against the design basis external flood mechanisms are provided in the USAR (Reference 1.7-4). The credited flood protection related attributes of the overall plant configuration that support the design for mitigation against external flooding have not changed from the time of initial licensing. Enhancements to procedural guidance supporting the implementation of protective actions against external flooding have been made over time. The temporary barriers deployed in Procedure 7.0.11 (Reference 1.7-13) now consist of engineered flood barriers. The use of these barriers, instead of the original sandbag barriers, has resulted in a significant reduction of the barrier deployment time and effort.

Changes to the hydrosphere around the CNS site and physical changes to the CNS site (e.g., security changes, buildings, etc.) are discussed in Section 1.4 of this report.

1.4 CHANGES TO THE WATERSHED OR TO THE LOCAL AREA

As shown in Figure 1.9-6, the river was much wider and further to the east than it is today as illustrated in Figure 1.9-7 and Figure 1.9-8 Reference 1.7-14).

The CNS site is located in the floodplain of the Missouri River at approximate Missouri River Mile 532.5. As a result of the location, the site is subjected to hydrometeorological events relevant to inland sites.

The Missouri River Basin upstream of CNS and downstream of Gavins Point Dam combined with Platte River Basin, as discussed in Section 2.2 of this report, encompasses approximately 132,657 mi^2.

Changes in watershed properties particularly affect the estimation of the PMF and upstream dam failure flooding. It is expected that watershed characteristics do change through the years, with the expansion of urban areas and change in land use and land cover. However, all of the changes that may have occurred in the past were captured in the current flooding hazard reevaluation. The reevaluation took into account the existing watershed conditions, which were incorporated through the hydrologic and hydraulic model calibration efforts. It should be noted that since the construction and operation of CNS, there has been no major upstream dam or impoundment on the Missouri River or tributaries constructed or proposed.

Any changes in local area that may have occurred in the past, including the vehicle barrier installed for a security measure, were incorporated in the LIP analysis of the current flooding hazard reevaluation. (See Section 2.1 of this report.)

1.5 CURRENT DESIGN BASIS FLOOD PROTECTION AND MITIGATION FEATURES

The general ground elevation surrounding CNS Class I structures, which corresponds to the PMF event elevation, is elevated 13 ft above the natural floodplain to 903 ft MSL (Reference 1.7-4, Chapter II, Section 4.2.2.2). The finished floor elevation of all Class I structures is placed at elevation 903.5 ft MSL, or 0.5 ft above the PMF event. These structures were designed for a hydraulic load equivalent to a groundwater elevation of 903 ft MSL and reviewed for integrity for a river elevation up to 906 ft MSL. Grade level openings on exterior walls of the buildings (except for the Intake Structure) are protected from wave effects and WSELs up to 906 ft MSL with flood barriers erected per CNS Procedures 7.0.11 (Reference 1.7-13) and 5.1, Flood (Reference 1.7-15), as previously discussed in Section 1.2.11.7 of this report.

The CNS Flood Protection Feature Inspections, in accordance with the request from the U.S. Nuclear Regulatory Commission (NRC) as defined in the Recommendation 2.3, Flooding, Enclosure 4 of the 50.54(f) letter dated March 12, 2012 (Reference 1.7-5) found that the CNS flood protection active and passive features, e.g., walls, floors, roofs, penetration seals, doors, check valves, etc., were confirmed to be installed per design, functional, in good material condition, and appropriately controlled procedurally to ensure continued functionality.

The following CNS safety-related plant structures and procedures have been identified as protected from flooding as noted:

Intake Structure

The maximum water level on the outside of the Intake Structure of 909.2 ft MSL (PMF plus wave effects) will not affect safe operation of the Service Water pump motors. The Intake Structure is built with a floor elevation of 903.5 ft MSL. The Service Water pump motors are positioned approximately 4.5 ft above the floor and are protected by 24-inch-thick concrete walls to an elevation of 919 ft MSL. Therefore, direct wave action will be dissipated and the water level in the room would be below the elevation of the Service Water pump motors (Reference 1.7-4, Chapter II, Section 4.2.2.2).

Reactor Building

The Reactor Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Emergency Diesel Generator Building

The Emergency Diesel Generator Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Radwaste Building

The Radwaste Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Control Building

The Control Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Controlled Corridor

The Controlled Corridor is protected from flood water by grade level Reactor and Turbine Building walls to elevation 906 ft MSL.

Turbine Building

The Turbine Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Z-Sump (Below ERP Tower)

As noted in the USAR (Reference 1.7-4, Chapter II, Section 4.2.2.2), the top of floor drain Z-Sump, at the base of the ERP tower, is located at elevation 891 ft MSL and, therefore, is within postulated flood levels. The Z-Sump contains equipment essential to the operation of the Standby Gas Treatment System (SGTS) and, therefore, the sump must remain functional whenever Secondary Containment is required. Although the ground elevation at the sump is only 890 ft MSL, the Z-Sump will not be affected by flooding since the sump penetrations are sealed and the proper functioning of the sump is monitored when flood levels reach 890 ft MSL.

Diesel Fuel Storage Tanks

As noted in the USAR (Reference 1.7-4, Chapter II, Section 4.2.2.2); there is sufficient fuel in the Diesel Oil Storage Tanks to ensure seven days of operation of a single diesel generator powering a single critical division of safe shutdown loads. This time duration is sufficient to obtain more fuel, if needed. The two storage tanks are buried and their appendages are protected by a substantial cover. The manholes providing access to the Diesel Oil Transfer Pumps, the capped fill connections, and the tank vents are all located above 906 ft MSL. The design and installation of the tanks ensure flotation does not occur when empty during the PMF.

The following CNS plant structures have been identified as important to the protection of the various safety-related structures and equipment and are also protected from flooding as noted:

Augmented Radwaste Building

The Augmented Radwaste Building is protected from wave effects and flood water by grade level building walls and by temporary flood barriers to elevation 906 ft MSL.

Boiler Room

The Boiler Room is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to elevation 906 ft MSL.

Fan Room

The Fan Room is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to elevation 906 ft MSL.

Water Treatment Plant

The Water Treatment Plant is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to elevation 906 ft MSL.

Tool Crib

The Tool Crib is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to elevation 906 ft MSL.

Machine Shop

The machine shop is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to the elevation of 906 ft MSL.

Multi-Purpose Facility (MPF) Building

The MPF Building is protected from wave effects and flood water by grade level building foundation walls and by temporary flood barriers to the elevation of 906 ft MSL.

Other Site Structures

Various site out-buildings and structures classified as not important to safe operation and shutdown of the plant are located at base elevations lower than the PMF elevation of 903 ft MSL. Per CNS Procedure 5.1, Flood (Reference 1.7-15), evaluation of the protection of these assets is determined by NPPD plant management by implementing actions based on various parameters, including plant safety, asset value, importance to overall long-term power generation, etc.

Procedures

Upon reaching an actual river level of 895 ft MSL or receiving a river level forecast of 902 ft MSL within the next 36 hours, the site flooding procedure is implemented. Apart from implementing temporary protection up to 906 ft MSL as previously discussed, the procedure directs the discharge of liquid radwaste as required to provide room for handling flood leakage. The floor and equipment drains and portable gasoline pumps are used to control leakage past the primary and secondary barriers.

A plant shutdown is initiated under any of the following conditions:

- Floodwaters either reach 902 ft MSL, or are forecast to reach 902 ft MSL within the next 36 hours (as from the 10,000 year flood or an upstream dam failure).
- Floodwater accumulates in either Diesel Generator Room, any of the four Reactor Building Quads, or the Control Building basement.
- Plant conditions warrant reactor to be shut down.

(Reference 1.7-4, Chapter II, Section 4.2.2.2)

1.6 ADDITIONAL SITE DETAIL

All available site details have been noted.

1.7 REFERENCES

1.7-1. U.S. Nuclear Regulatory Commission, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, March 12, 2012.

1.7-2. Safety Evaluation of the Cooper Nuclear Station. Docket 50-298. 02-14-73.

1.7-3. NPPD Engineering Evaluation Number 12-035, Revision 0, "Review of 2012 Topographic Survey of CNS," March 2013.

1.7-4. Cooper Nuclear Station USAR, Revision xxvi7.

1.7-5. Nebraska Public Power District, Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force 10 CFR 50-54 (f) Section 2.3 Flooding Response.

1.7-6. Nebraska Public Power District, Columbus, Nebraska, Engineering Criteria Document for Cooper Nuclear Station. June, 3, 1970.

1.7-7. Calculation 2012-12283, Revision 0, Evaluation of Local Probable Maximum Precipitation (PMP).

1.7-8. United States Coast Guard, Mississippi River and Tributaries Waterways Action Plan Missouri River Annex 2011.

1.7-9. Cooper Nuclear Station, FSAR Question Number 2.1, Amendment Number 9.

1.7-10. Hydrometeorological Report No. 33, Dated April 1956.

1.7-11. Cooper Nuclear Station, FSAR Question Number 2.3.6, Amendment 17.

1.7-12. U.S. Nuclear Regulatory Commission, "Interim Staff Guidance for Performing the Integrated Assessment for External Flooding," JLD-ISG-2012-05, November 30, 2012.

1.7-13. CNS Operations Manual, Maintenance Procedure 7.0.11, Flood Control Barriers, Revision 29, Dated 02-19-14.

1.7-14. Calculation 2014-00225, Revision 0, Task 810 – Missouri River Channel Geomorphic Evaluation at CNS.

1.7-15. CNS Emergency Procedure 5.1 Flood, Revision 13.

1.7-16. Hydrodynamic Computer Modeling to Predict Missouri River Probable Maximum Flood Elevations and Flow Velocities at Cooper Nuclear Station, NEDC 11-076, Revision 1, Dated August 08, 2011.

1.7-17. ArcMap Version 9.3.1 (Build 3000). ESRI, Redlands, CA.

1.8 TABLES

The tables associated with Section 1 are presented on the following pages.

Table 1.8-1: Missouri River Dams

Name	Location Near	River Mile
Gavins Point	Yankton, South Dakota	811
Fort Randall	Lake Andes, South Dakota	880
Big Bend	Chamberlain, South Dakota	987
Oahe	Pierre, South Dakota	1072
Garrison	Bismarck, North Dakota	1389
Fort Peck	Glasgow, Montana	1771

Note:
1. Plant location: River Mile 532.5

Table 1.8-2: Design Evaluation Summary

Condition	CNS Evaluation	Comment
PMF Peak Flow Rate (1)	600,000 cfs	PMP Event centered above Brownville, NE.
PMF Still Water Surface Elevation	903 ft MSL (2)	Plant site elevated ±903 ft MSL. First floor of all Class I buildings at 903.5 ft MSL.
PMF + Wave Runup at Intake Structure	909.2 ft MSL	Flood barriers not needed at intake since critical equipment is elevated above 906 ft MSL and protected from wave impacts.
PMF + Wave Action Main Plant Complex	No impact	Class I buildings' grade level openings at 903.5 ft MSL protected by flood barriers from standing water up to 906 ft MSL.

Notes:
1. The USACE estimated a higher river stage during the PMF than the AEC, despite the fact it used a lower flow rate (600,000 cfs versus 1,000,000 cfs). Details of the methodology utilized by the AEC to develop its stage flow relationship were not described, except for a statement in the SER text that the AEC constructed an "Analytical Model". The analysis conducted by CNS in 2011 is consistent with the USACE stage discharge estimates (Reference 1.7-16).
2. This elevation is based on a steady flow rate of 600,000 cfs. See Reference 1.7-4, Chapter II, Section 4.2.2.1.
3. See Reference 1.7-5, Section 2.7.

Table 1.8-3: Missouri River Drainage Area

Location (Missouri River)	Drainage Area (Approximate (Square Miles)
Above Fort Peck Dam	57,500
Above Garrison Dam	181,400
Above Oahe Dam	243,490
Above Fort Randall Dam	263,480
Above Gavins Point Dam	279,480
Above Mouth of Platte River	323,530
Platte River at Mouth	90,200
At Cooper Nuclear Station	414,600

Reference 1.7-4, Chapter II, Section 4.2.2.1

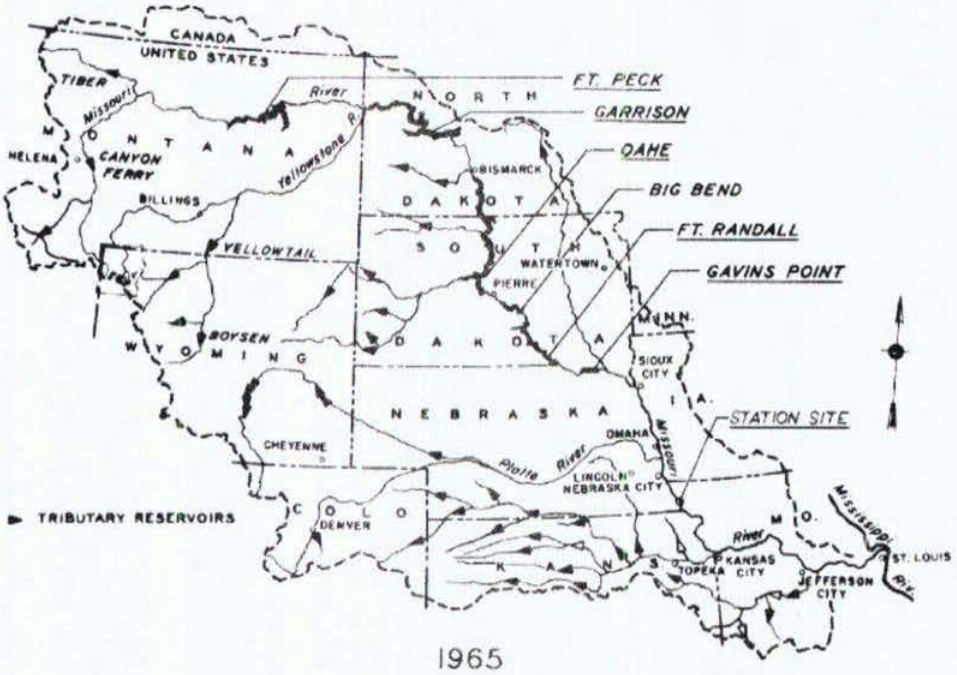
1.9 FIGURES

The figures associated with Section 1 are presented on the following pages.

Figure 1.9-1: Site Location

Reference 1.7-17

Figure 1.9-2: Missouri River Tributaries



Reference 1.7-4, Figure II-4-1

Figure 1.9-3: Overhead View of CNS 1

Reference 1.7-17

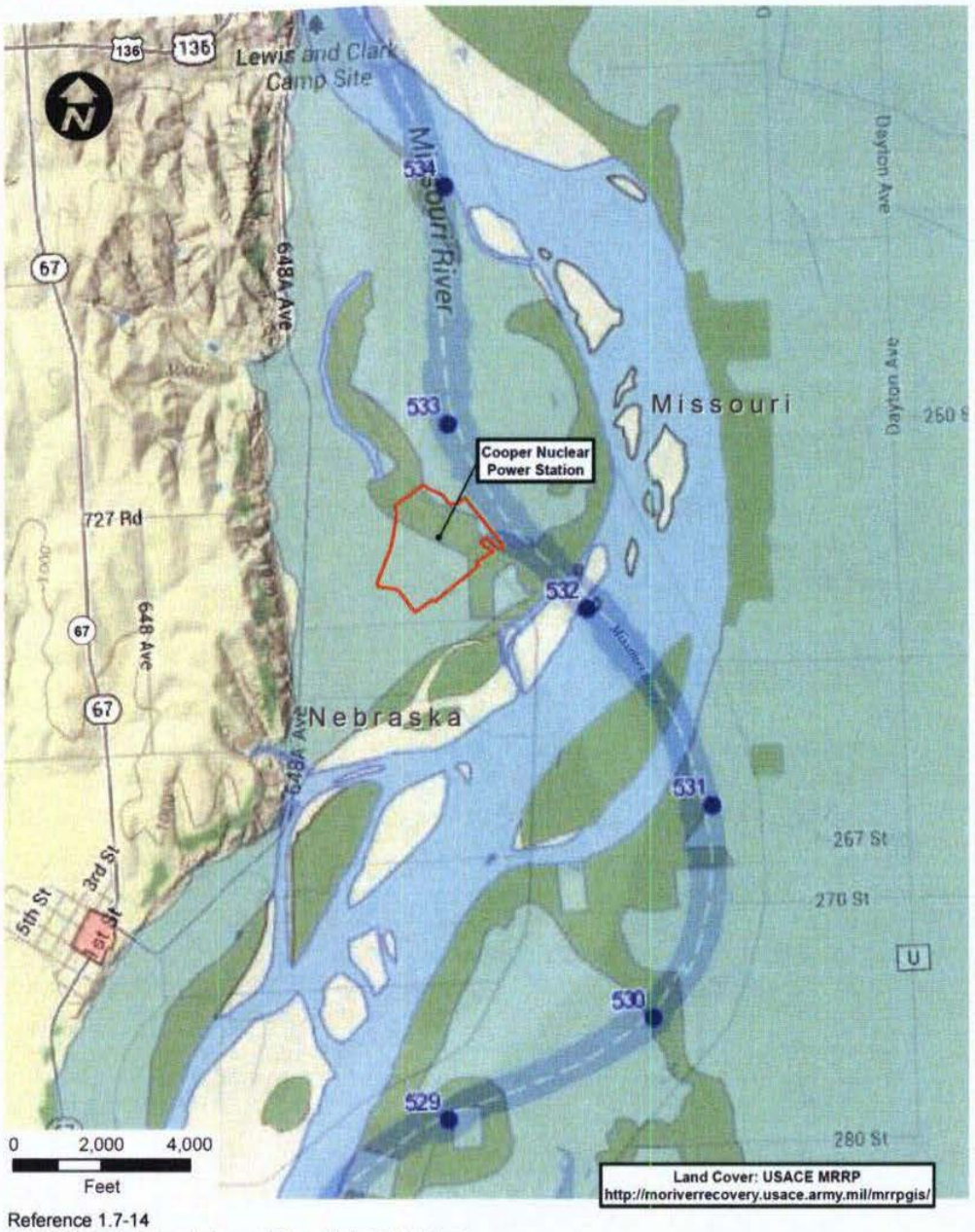
Figure 1.9-4: Overhead View of CNS 2

Reference 1.7-17

Figure 1.9-5: Missouri River Reservoir System

Reference 1.7-8

Figure 1.9-6: Missouri River Path in 1879 at the Site of CNS



Reference 1.7-14
(Note: Current river path is overlaid and is in faded blue.)

Figure 1.9-7: Aerial Photograph of the Plant Site in 1971

Reference 1.7-14 (This photograph was taken after construction of CNS began in 1968, but prior to CNS officially opening in 1974. The river channel and banks looked much like they do today.)

Figure 1.9-8: Aerial Photograph of CNS Showing Flooding

Reference 1.7-14 (This Figure shows a satellite image of the Missouri River taken on July 9, 2011 and the effect of the maximum releases from Gavins Point Dam during the 2011 flood. Flood waters had inundated the floodplain in many of the areas where the Missouri River had once meandered within the boundaries of the levees. There was visible flooding east of the eastern levee due to the upstream levee breaches. There was also visible standing water on the west side of the river, west of the levee, likely due to interior drainage. There is no evidence that a new channel was beginning to form. The resolution of the image is such that it is not possible to discern the status of the bank stabilization structures.)

2. FLOOD HAZARD REEVALUATION

2.1 LOCAL INTENSE PRECIPITATION (LIP)

Probable Maximum Precipitation (PMP), also known as Local Intense Precipitation (LIP), is the measure of the extreme precipitation (high intensity/short duration) at a given location. Evaluation of the flooding hazards to the safety-related Systems, Structures, and Components (SSCs) at Cooper Nuclear Station (CNS) due to LIP is presented in this section of the report. U.S. Nuclear Regulatory Commission (NRC) guidelines NUREG/CR-7046 (Reference 2.1-1), NRC Regulatory Guide 1.59 (Reference 2.1-2), and ANSI/ANS-2.8-1992 (Reference 2.1-3) form the basis for the approach and methodology used in this evaluation. The analysis was performed with the conservative assumption that the local storm drainage system (culverts, ditches, storm sewers, dry wells, etc.) would not be functional during the LIP event.

The LIP flood effects on CNS were determined by performing site-specific hydrologic and hydraulic analyses. The rational method (Reference 2.1-4) with conservative runoff coefficients (i.e.,1.0) was used for computation of peak PMP runoff from different drainage areas of the plant site. The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) computer program (Reference 2.1-5) was used to determine the maximum water surface elevation and flow velocity. Due to the relatively short duration of the flooding event and the shallow depth of inundation, the effects of wind-waves were not considered. Reference 2.1-6 defines Plant Datum as the same as Datum used in the USAR (which is MSL).

The vertical datum used for this section (i.e., Section 2.1) was the CNS Plant Datum. The National Geodetic Vertical Datum of 1929 elevation (NGVD29) is equal to Plant Datum elevation plus 0.11 feet [ft] (Reference 2.1-6). The North American Vertical Datum of 1988 elevation (NAVD88) is equal to NGVD29 elevation plus 0.26 ft (Reference 2.1-6). The NAVD88 elevation is equal to Plant Datum elevation plus 0.37 ft. All directions are with respect to the Plant North direction, which is oriented to the west of the true north direction.

The effect of LIP on CNS was not evaluated previously in the Updated Safety Analysis Report (USAR, Reference 2.1-7). As presented in Section 4.2.2.2 of the USAR, the finished floor elevation of Principle Class 1 Structures (SSCs) is at 903.5 ft Plant Datum (903.87 ft NAVD88) and the general site grade elevation is at 903.0 ft Plant Datum (903.37 ft NAVD88).

2.1.1 Probable Maximum Precipitation Depths

As prescribed in NUREG/CR-7046 (Reference 2.1-1), the LIP used in the analysis is the 1-hour, 1-square mile PMP at the CNS site. Parameters to estimate the LIP were obtained from the U.S. National Weather Service (NWS) Hydrometeorological Reports HMR 51 and HMR 52 (see References 2.1-8 and 2.1-9).The estimated depths from HMR 51 are for precipitation durations 6 hours and longer and drainage areas from 10 to 20,000 square miles. These estimates were used with procedures outlined in HMR 52 to determine PMP depths of various durations less than 1 hour and

drainage areas up to 1 square mile for the site. Table 2.1-1 presents the PMP depths and intensities for various durations at the CNS site.

2.1.2 Drainage Areas and Local Drainage Parameters

The CNS site is located on the west (right) bank of the Missouri River near Brownville, Nebraska, at approximate River Mile (RM) 532.5. The latest available topographic survey (Reference 2.1-6) for the plant site area was used for delineation of drainage areas (zones). The main plant area is relatively flat without well-defined flow paths, and is delineated into drainage areas considering drainage away from the Central Building Complex in all directions. The majority of the plant area perimeter is bounded by 9 to 10 rows of jersey barriers (Figure 2.1-1) with variable longitudinal spacing (1 inch to 36 inches). A portion of the security barrier south of the plant is formed by Kontek concrete block barriers. The jersey barriers are 12.5 ft long and 32 inches (in) high and the Kontek blocks are 10 ft long and 3 ft high. Due to the variability in opening size, potential for barrier relocation, and potential for staggered openings between barrier rows; conservatively, openings between barriers were not considered flow paths for this analysis.

Figure 2.1-2 shows the four drainage areas considered in the main plant area, identified as Zones A, B, C, and D. All safety-related facilities at CNS are located within these four zones. Peripheral portions of the main plant area that are not directly connected to, or that do not affect the safety-related facilities, were not included in the PMP zones. These small areas located along the periphery of the main plant area would generate a relatively small amount of runoff that would drain away from these areas without affecting the runoff draining away from the safety-related facilities.

Zone A is located on the east side of the plant, near the Missouri River. The Intake Structure is located along the downstream (eastern) boundary of Zone A. The grade elevation in this zone varies from 902 ft (902.37 ft NAVD88) to 903.5 ft (903.87 ft NAVD88). Although runoff could flow to the south where the grade slopes down toward the area of the discharge canal and sludge pond, all runoff from Zone A is considered to flow over the eastern boundary to the river on the south side of the Intake Structure, resulting in conservative estimates for the maximum water level in Zone A.

Zone B is located on the north side of the main plant area. Security barriers are located along the northern edge of the zone at the downstream boundary, with relatively high elevations on the west side and lower elevations on the east side toward the river. The grade elevations at the downstream boundary of Zone B range from 902.5 ft (902.87 ft NAVD88) on the west side, to 897 ft (897.37 ft NAVD88) and below on the east side. Due to multiple obstructions on the western side of Zone B (i.e., tanks, buildings, etc.), the flow paths on the west side of some cross sections were considered blocked and not used for conveyance. Additionally, due to the relatively high grade elevation and security barriers downstream of the tanks, the areas between the tanks were designated as ineffective flow areas within the HEC-RAS model. All runoff from Zone B was considered to drain away from the main plant through the area between the Condensate Storage Tank and the Fabrication Shop, and discharge over the top of the security barriers on the east side of the northern boundary. Overtopping of the barriers was considered at the downstream boundary of Zone B.

Zone C is located on the south side of the main plant area. Runoff from this zone drains over the southern periphery of the main plant area to the low elevation area near the discharge canal, where it can flow unobstructed to the Missouri River. The grade elevations at the downstream boundary of Zone C range from 904 ft (904.37 ft NAVD88) to 902 ft (902.37 ft NAVD88), and all runoff from Zone C was considered to exit the zone on the west side of the storage tanks and buildings located near the downstream (southern) boundary.

Zone D is located on the west side of the main plant area. The security barriers (32 inches high) are located at the western edge of this zone and act as an obstruction. As runoff accumulates behind the barriers and causes ponding in Zone D, runoff will flow to the south. Overtopping of the barriers along the western boundary of Zone D was not considered in this analysis due to the relatively high elevations of the top of barriers. Although the flow path along the west side of the East Warehouse between the security barriers and the building has the capacity to convey the entire peak flow from Zone D, it is expected that some runoff will flow into Zones B and C. Flow-balancing calculations were performed to determine the quantities of runoff flowing to these zones.

The offsite drainage area was delineated (Figure 2.1-3) considering Federal Levee R548 that is adjacent to CNS with a top elevation of approximately 902 ft (902.37 ft NAVD88). The levee extends north of the plant, where it terminates at the higher ground near the bluffs. South of CNS, the levee follows the right bank of the Missouri River, curving around to the west, running parallel to the left bank of the Nemaha River, and turning north where it meets the higher ground near the town of Nemaha. The area enclosed by the levee and the higher bluffs to the west measures 4000 acres (ac). The effect of runoff from the offsite drainage area on PMP water levels near CNS is discussed in Section 2.1.4.

2.1.3 Peak Discharges

To compute the peak runoff, time of concentration for each zone was estimated using the Kerby Equation (Reference 2.1-10). For each zone, flow length, average slope, and roughness coefficient were estimated and used as input for the Kerby Equation. The surfacing in the main plant area is mostly gravel and concrete or asphalt-paved, and the roughness coefficient was considered to be 0.02 (Reference 2.1-4).The estimated times of concentration are presented in Table 2.1-2. Using the PMP values corresponding to the time of concentration, the applicable PMP intensity for each zone was estimated. Employing the rational method (Reference 2.1-4), peak runoff for each of the zones was estimated. A conservative runoff factor value of 1.0 (indicating 100% runoff without any infiltration or other losses) was used in the rational method.

2.1.4 Hydraulic Model Setup

For computations of water levels, the HEC-RAS Version 4.1 computer program (Reference 2.1-5) was used. Separate HEC-RAS runs for each zone were performed. The site features and flow obstructions in each drainage area are characterized by cross sections (Figure 2.1-4 through Figure 2.1-7) and used in the hydraulic analysis.

The surfacing in the main plant area is primarily gravel and concrete or asphalt-paved. Reference 2.1-4 recommends Manning's roughness coefficients ranging from 0.011 to 0.025 for such surfaces. Conservatively, a Manning's roughness coefficient of 0.030 was used to account for minor obstructions.

The flow at each cross section was computed by prorating the peak discharge of the entire zone based on the drainage area upstream of that cross section. Table 2.1-3 through Table 2.1-6 present the peak flow at each cross section for Zones A through D, respectively.

For Zones A, C, and D, the critical depth was considered as the downstream boundary condition to account for the flow transition from the relatively flat main plant area, over the peripheral boundary and down a steep slope. Critical depth was considered for the downstream boundary condition in Zone B to account for the flow over the security barriers, which were considered to act as a weir. The present-day regulatory guidance (References 2.1-1 and 2.1-11) does not provide combined-events criteria governing the coincident water level in an adjacent river during the LIP event. The normal river water level at CNS is well below elevation 890 ft (880.37 ft NAVD88), and the CNS USAR defines the normal summer water level as 880 ft (890.37 ft NAVD88 [Reference 2.1-7]).The 25-year water level in the Missouri River at CNS is 898.04 ft (898.41 ft NAVD88 [Reference 2.1-12]).The 25-year water level is well below the lowest elevation of any cross section considered in the evaluation and will not affect drainage from the site during an LIP event. Therefore, there is no backwater effect from the Missouri River at the downstream boundary for Zones A, B, and C. Zone D discharges to the floodplain behind the USACE levee, and is not affected by water levels in the Missouri River or the offsite area.

Runoff from the offsite drainage area will pond on the floodplain behind the USACE levee. Comparing the total PMP runoff volume with the storage volume available behind the levee showed that the maximum surface elevation of the ponded water level will be less than 892 ft (892.37 ft NAVD88).Therefore, water levels at the boundary of the main plant area, during an LIP event, will not be affected by runoff from the offsite drainage area.

Several temporary trailers are installed in the main plant area at CNS. The trailers are accounted for in the analysis as complete obstructions. Sensitivity runs with and without the trailers performed using HEC-RAS indicated that these trailers have a negligible effect on the PMP water levels estimated for this analysis.

Initially, the maximum water levels for each zone were computed without considering the cross flow between the adjacent zones. Subsequently, a trial-and-error flow-balancing exercise was performed to determine the cross flows that result in matching water levels at the interface between zones.

2.1.4.1 LIP Runoff Water Levels without Cross Flow between Zones

The resulting maximum water level in each zone during an LIP event, without consideration of cross flow between zones, is summarized below:

- Zone A maximum water level = 903.05 ft (903.42 ft NAVD88)
- Zone B maximum water level = 903.35 ft (903.72 ft NAVD88)
- Zone C maximum water level = 903.59 ft (903.96 ft NAVD88)
- Zone D maximum water level = 903.80 ft (904.17 ft NAVD88)

2.1.4.2 Water Levels with Cross Flow between Zones

Preliminary water level estimates without consideration of cross flow showed the water levels in Zones B and D differ by nearly 6 inches. Through a process of trial-and-error, a cross flow of 63 cubic feet per second (cfs) is considered to flow from Zone D to Zone B (Figure 2.1-5). A cross flow of 63 cfs was chosen because it represents all of the peak flow at the upstream-most cross section of Zone D and provides closely matching (less than a 1-inch difference) final water surface elevations between Zones B and D.

Cross flow can occur from Zone D to Zone C upstream of the East Warehouse, and from Zone C to Zone D downstream of the East Warehouse (Figure 2.1-6). Cross flow from Zone D into Zone B was deducted from Zone D before the cross flow balancing between Zones C and D was performed. Cross flow from Zone C into Zone A (Figure 2.1-7) was estimated following the cross flow balancing between Zones C and D.

The maximum water level in Zone B is approximately 5 inches higher than the maximum water level in Zone A. The broad-crested weir equation (Reference 2.1-13) was used to estimate the cross flow from Zone B to Zone A. The potential for submerged weir flow was checked using Figure 5-15 of Reference 2.1-14. A total cross flow of 108 cfs enters Zone A at the upstream boundary and was applied to every cross section in the zone. Conservatively, the flows at cross sections in Zone B and Zone C were not reduced to account for this flow out of the zone. Figure 2.1-8 through Figure 2.1-10 present the results of the hydraulic analysis for all HEC-RAS cross sections considering cross flow between two adjacent zones. The resulting maximum water level in each zone during an LIP event, considering cross flow between zones, is summarized below.

- Zone A maximum water level = 903.19 ft (903.56 ft NAVD88)
- Zone B maximum water level = 903.47 ft (903.84 ft NAVD88)
- Zone C maximum water level = 903.50 ft (903.87 ft NAVD88)
- Zone D maximum water level = 903.50 ft (903.87 ft NAVD88)

2.1.4.3 Velocity

The maximum velocity obtained from the HEC-RAS output for each zone is summarized below.

- Zone A maximum velocity = 3.64 feet per second (fps)
- Zone B maximum velocity = 4.49 fps
- Zone C maximum velocity = 2.87 fps
- Zone D maximum velocity = 5.02 fps

The HEC-RAS simulations were performed under subcritical flow conditions; however, the results indicated that there could be supercritical flow conditions in Zone B and Zone D. Additional HEC-RAS test runs using a mixed-flow regime resulted in no change to the maximum water levels estimated for these zones. Since the subcritical flow dominates in the upstream areas, upstream water levels are controlled by water levels downstream, not the upstream boundary condition. However, the velocities at some cross sections are affected when a mixed-flow regime is employed.

In Zone D, supercritical flow occurs in the area located between the Warehouse and the security barriers at Cross Section D500, with a maximum velocity of 5.12 fps. The model indicated that a hydraulic jump occurs between Cross Sections D500 and D400. The Froude number is close to 1.0, indicating that the hydraulic jump is weak; however, the potential for erosion exists due to turbulence at the location of the jump. In Zone B, supercritical flow occurs on the steep slope at the periphery of the main plant area at Cross Section B200. The velocity at this location is 9.34 fps; hence, there is a potential for erosion of the sloped surface. Both of the locations that have potential for site erosion are located around the periphery of the plant, away from safety-related SSCs, and would not affect any CNS safety-related facilities.

2.1.5 Effect of LIP

Table 2.1-7 presents a summary of the Maximum Water Levels in all zones at different cross sections. These water levels consider cross flow between zones for steady-state, subcritical flow conditions. The maximum estimated water level due to LIP at CNS is 903.50 ft (903.87 ft NAVD88), which is equal to the finished floor elevations for Principal Class 1 Structures. While significant inundation above the finished floor elevations is not expected, the potential for minor flooding at the entrances of these structures may exist during the peak of the event.

Reference 2.1-14 recommends a maximum permissible flow velocity of 6.0 fps to prevent erosion of channels with fine-gravel surfacing. The potential for erosion in the main plant area is low, as the velocities do not exceed 6 fps and are generally less than 3 fps. Velocities on the steep slope at the periphery of the main plant area may cause local erosion of the sloped surface, but there would be no effect on safety-related facilities.

Due to the relatively short duration of the flooding event and the shallow depth of inundation, the effects of wind-waves were not considered. Additionally, debris loading and transportation were not considered in the analysis due to the relatively low velocity and shallow depth of LIP flood waters near CNS safety-related facilities, and due to the lack of debris sources in the CNS main plant area.

2.1.6 References

2.1-1. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

2.1-2. Regulatory Guide 1.59, Design Basis Floods for Nuclear Power Plants, Revision 2, Office of Standards and Development, U.S. Nuclear Regulatory Commission, August 1977.

2.1-3. American Nuclear Society, ANSI/ANS-2.8-1992, Determining Design Basis Flooding at Power Reactor Sites, July 1992.

2.1-4. Handbook of Applied Hydrology, VenTe Chow, McGraw - Hill Inc., 1964.

2.1-5. Hydrologic Engineering Center's River Analysis System (HEC-RAS) Version 4.1 for Windows. U.S. Army Corps of Engineers, 2010.

2.1-6. NPPD Engineering Evaluation Number 12-035, Revision 0, "Review of 2012 Topographic Survey of CNS," March 2013.

2.1-7. Updated Safety Analysis Report (USAR - 2.7), Cooper Nuclear Station, Site and Environs, Hydrology, Site Flooding Protection, Dated 01/16/01.

2.1-8. Hydrometeorological Report No. 51 (HMR 51), Probable Maximum Precipitation estimates, United States East of the 105th Meridian, National Weather Service, National Oceanic and Atmospheric Administration (NOAA), June 1978.

2.1-9. Hydrometeorological Report No. 52 (HMR 52), Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, National Weather Service, National Oceanic and Atmospheric Administration (NOAA), August 1982.

2.1-10. Hydrology and Hydraulic Systems, Ram S. Gupta, Second Edition, Waveland Press, Inc., 2001.

2.1-11. Standard Review Plan, Revision 4, NUREG-0800, Section 2.4.2 Floods, United States Nuclear Regulatory Commission (USNRC), March 2007.

2.1-12. Upper Mississippi River System Flow Frequency Study, Hydrology and Hydraulics, Appendix F, Missouri River, U.S. Army Corps of Engineers, Omaha District, November 2003.

2.1-13. Handbook of Hydraulics, Ernest F. Brater, Horace W. King, Sixth Edition, McGraw-Hill, Inc., 1976.

2.1-14. Engineering Manual EM 1110-2-1601, "Engineering and Design – Hydraulic Design of Flood Control Channels," Table 2-5, p 2-16, U.S. Army Corps of Engineers, 1991.

2.1.7 Tables

Tables associated with Section 2.1 are presented on the following pages.

Table 2.1-1: PMP Values and Intensities at the CNS Site

PMP Duration and Area	1 hr, Point Location Ratio	Source	PMP Depth (in)	PMP Intensity (in/hr)
24 hr, 10 mi^2	-	HMR 51 - Figure 20	33.3	-
1 hr, point location	-	HMR 52 - Figure 24	18.2	18.2
30 min, point	0.765	HMR 52 - Figure 38	13.9	27.8
15 min, point	0.534	HMR 52 - Figure 37	9.7	38.8
5 min, point	0.338	HMR 52 - Figure 36	6.2	74.4

Table 2.1-2: Peak Discharge

Zone	Drainage Area (acres)	Flow Length (ft)	Slope (ft/ft)	Time of Concentration (min)	Rainfall Intensity (in/hr)	Peak Discharge (cfs)
A	2.61	130	0.0077	4.0	74.4	194
B	5.30	280	0.0036	6.9	59.7	316
C	4.00	285	0.0035	7.0	59.1	236
D	6.29	650	0.0031	10.6	46.2	291

Note:
1. A 5-minute time of concentration is applied to any drainage area in which the estimated time of concentration is less than 5 minutes.

Table 2.1-3: Drainage Area and Peak Flow at each Cross Section – Zone A

Column:	(1)	(2)	(3)	(4)
Cross Section	**Upstream Area (ac)**	**Ratio Upstream Area to Total Area**	**Peak Flow (cfs) Flow Profile: Initial (Without Cross Flow)**	**Peak Flow (cfs) Flow Profile: Final (With Cross Flow)**
A800	1.04	0.398	77	185
A700	1.22	0.467	91	199
A600	1.47	0.563	109	217
A500	1.73	0.663	129	237
A400	1.98	0.759	147	255
A300	2.23	0.854	166	274
A200	2.46	0.943	183	291
A100	2.61	1.000	194	302

Notes:
1. Column (2) = Column (1) / Total Area; Total Area = 2.61 ac
2. Column (3) = Column (2) x Peak Flow; Peak Flow = 194 cfs
3. Column (4) = Column (3) + 108 cfs from Zone B and Zone C (Cross Flow)

Table 2.1-4: Drainage Area and Peak Flow at each Cross Section – Zone B

Column:	(1)	(2)	(3)	(4)
Cross Section	**Upstream Area (ac)**	**Ratio Upstream Area to Total Area**	**Peak Flow (cfs) Flow Profile: Initial (Without Cross Flow)**	**Peak Flow (cfs) Flow Profile: Final (With Cross Flow)**
B1200	0.75	0.142	45	108
B1100	1.16	0.219	69	132
B1000	1.49	0.281	89	152
B900	2.13	0.402	127	190
B800	2.38	0.449	142	205
B700	2.81	0.530	168	231
B600	3.41	0.643	203	266
B500	3.65	0.689	218	281
B400	4.10	0.774	244	307
B300	4.65	0.877	277	340
B200	4.95	0.934	295	358
B100	5.30	1.000	316	379

Notes:
1. Column (2) = Column (1) / Total Area; Total Area = 5.30 ac
2. Column (3) = Column (2) x Peak Flow; Peak Flow = 316 cfs
3. Column (4) = Column (3) + 63 cfs from Zone D (Cross Flow)

Table 2.1-5: Drainage Area and Peak Flow at each Cross Section – Zone C

Column:	(1)	(2)	(3)	(4)
Cross Section	**Upstream Area (ac)**	**Ratio Upstream Area to Total Area**	**Peak Flow (cfs) Flow Profile: Initial (Without Cross Flow)**	**Peak Flow (cfs) Flow Profile: Final (With Cross Flow)**
C1200	1.08	0.270	64	65
C1100	1.19	0.298	70	72
C1000	1.37	0.343	81	84
C900	1.74	0.435	103	109
C800	2.11	0.528	124	130
C700	2.50	0.625	148	154
C600	2.69	0.673	159	165
C500	2.95	0.738	174	180
C400	3.26	0.815	192	170
C300	3.48	0.870	205	154
C200	3.82	0.955	225	145
C100	4.00	1.000	236	156

Notes:
1. Column (2) = Column (1) / Total Area; Total Area = 4.00 ac
2. Column (3) = Column (2) x Peak Flow; Peak Flow = 236 cfs
3. Column (4) = final profile resulting from cross flow balancing with Zone D

Table 2.1-6: Drainage Area and Peak Flow at each Cross Section – Zone D

Column:	(1)	(2)	(3)	(4)
Cross Section	**Upstream Area (ac)**	**Ratio Upstream Area to Total Area**	**Peak Flow (cfs) Flow Profile: Initial (Without Cross Flow)**	**Peak Flow (cfs) Flow Profile: Final (With Cross Flow)**
D2000	1.36	0.216	63	0
D1900	1.66	0.264	77	14
D1800	1.81	0.288	84	21
D1700	2.18	0.347	101	38
D1600	2.38	0.378	110	47
D1500	2.82	0.448	130	67
D1400	3.42	0.544	158	95
D1300	3.96	0.630	183	120
D1200	4.12	0.655	191	126
D1100	4.23	0.672	196	131
D1000	4.40	0.700	204	137
D900	4.78	0.760	221	152
D800	4.94	0.785	229	160
D700	5.19	0.825	240	171
D600	5.30	0.843	245	176
D500	5.46	0.868	253	184
D400	5.66	0.900	262	222
D300	5.82	0.925	269	258
D200	6.08	0.967	281	298
D100	6.29	1.000	291	308

Notes:
1. Column (2) = Column (1) / Total Area; Total Area = 6.29 ac
2. Column (3) = Column (2) x Peak Flow; Peak Flow = 291 cfs
3. Column (4) = final profile resulting from cross flow balancing with Zone C

Table 2.1-7: PMP Water Levels

Zone A Cross Section	Water Level (ft)	Zone B Cross Section	Water Level (ft)	Zone C Cross Section	Water Level (ft)	Zone D Cross Section	Water Level (ft)
A800	903.19	B1200	903.47	C1200	903.50	D2000	903.50
A700	903.17	B1100	903.46	C1100	903.48	D1900	903.50
A600	903.16	B1000	903.45	C1000	903.49	D1800	903.50
A500	903.14	B900	903.40	C900	903.47	D1700	903.50
A400	903.07	B800	903.39	C800	903.45	D1600	903.50
A300	903.02	B700	903.35	C700	903.44	D1500	903.50
A200	902.84	B600	903.33	C600	903.43	D1400	903.50
A100	902.55	B500	903.32	C500	903.43	D1300	903.49
-	-	B400	903.17	C400	903.42	D1200	903.49
-	-	B300	902.54	C300	903.38	D1100	903.49
-	-	B200	900.51	C200	903.15	D1000	903.47
-	-	B100	899.79	C100	902.30	D900	903.15
-	-	-	-	-	-	D800	903.06
-	-	-	-	-	-	D700	902.85
-	-	-	-	-	-	D600	902.51
-	-	-	-	-	-	D500	902.16
-	-	-	-	-	-	D400	902.02
-	-	-	-	-	-	D300	902.01
-	-	-	-	-	-	D200	901.76
-	-	-	-	-	-	D100	901.51

Note:
1. Water levels are all in Plant Datum; add 0.37 to convert to ft NAVD88.

2.1.8 Figures

Figures associated with Section 2.1 are presented on the following pages.

Figure 2.1-1: Location of Security Barriers

Figure 2.1-2: Drainage Areas for LIP Evaluation

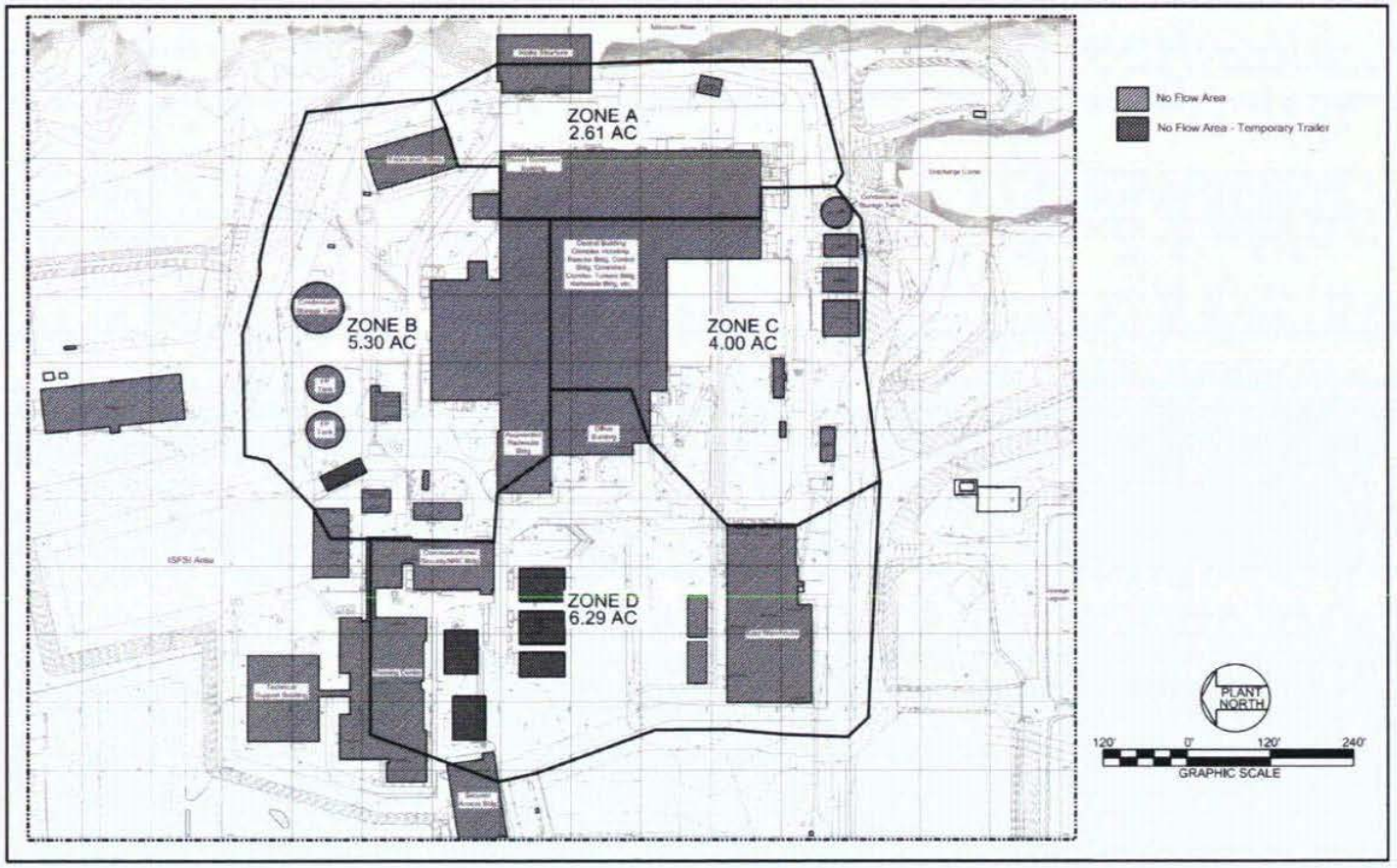


Figure 2.1-3: Offsite Drainage Area behind USACE Levee

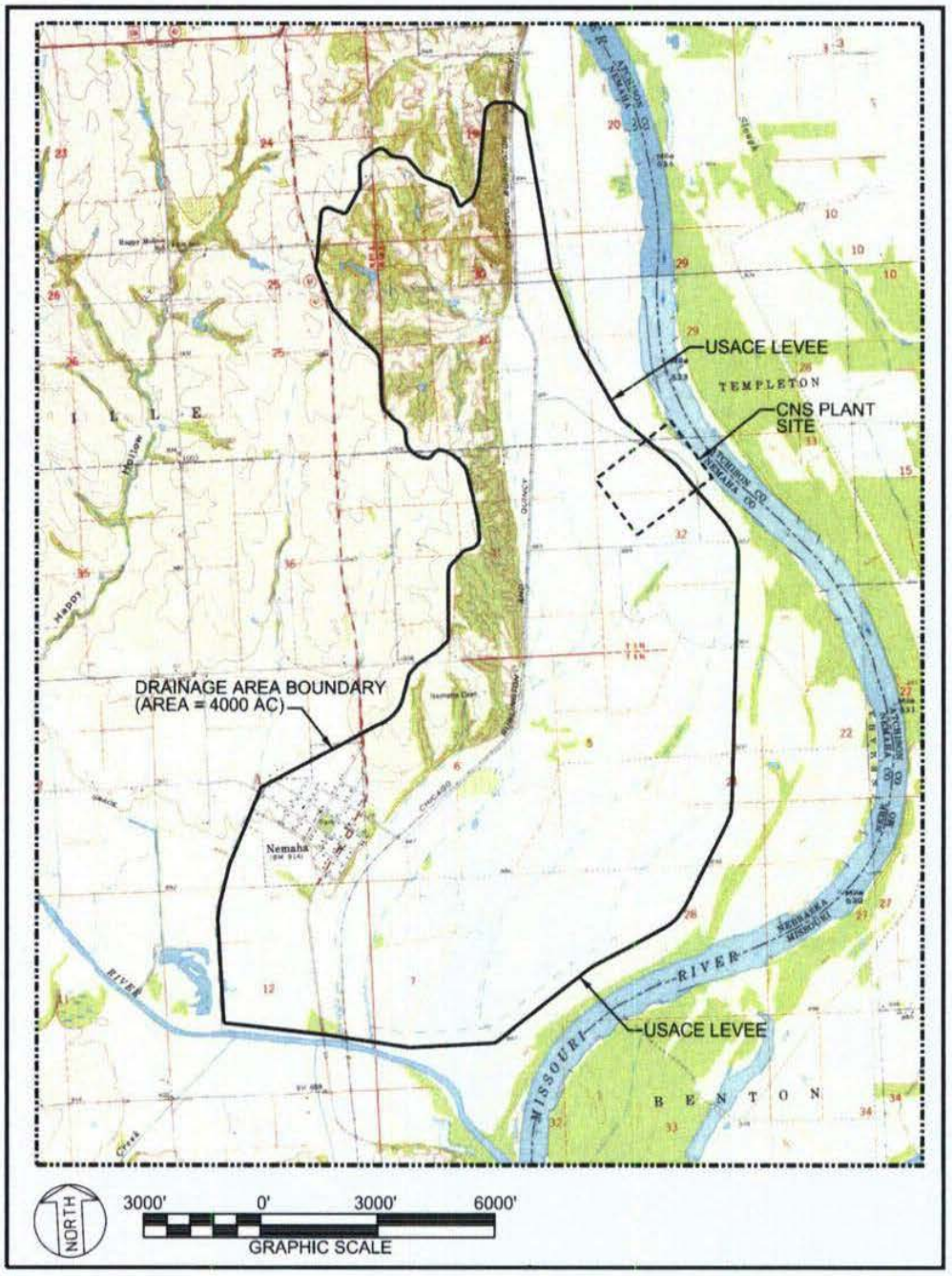


Figure 2.1-4: Cross Sections for LIP Evaluation - Zone A

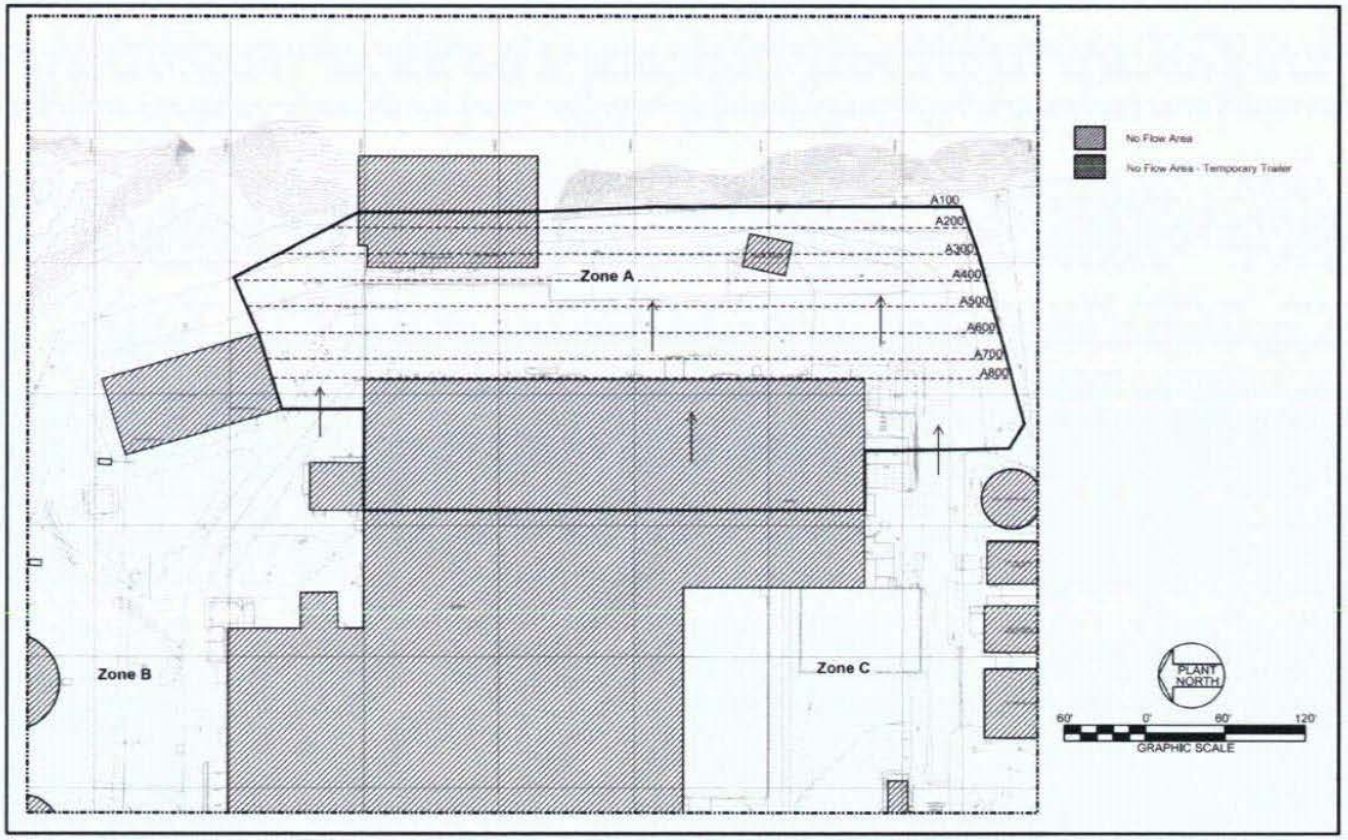


Figure 2.1-5: Cross Sections for LIP Evaluation - Zone B

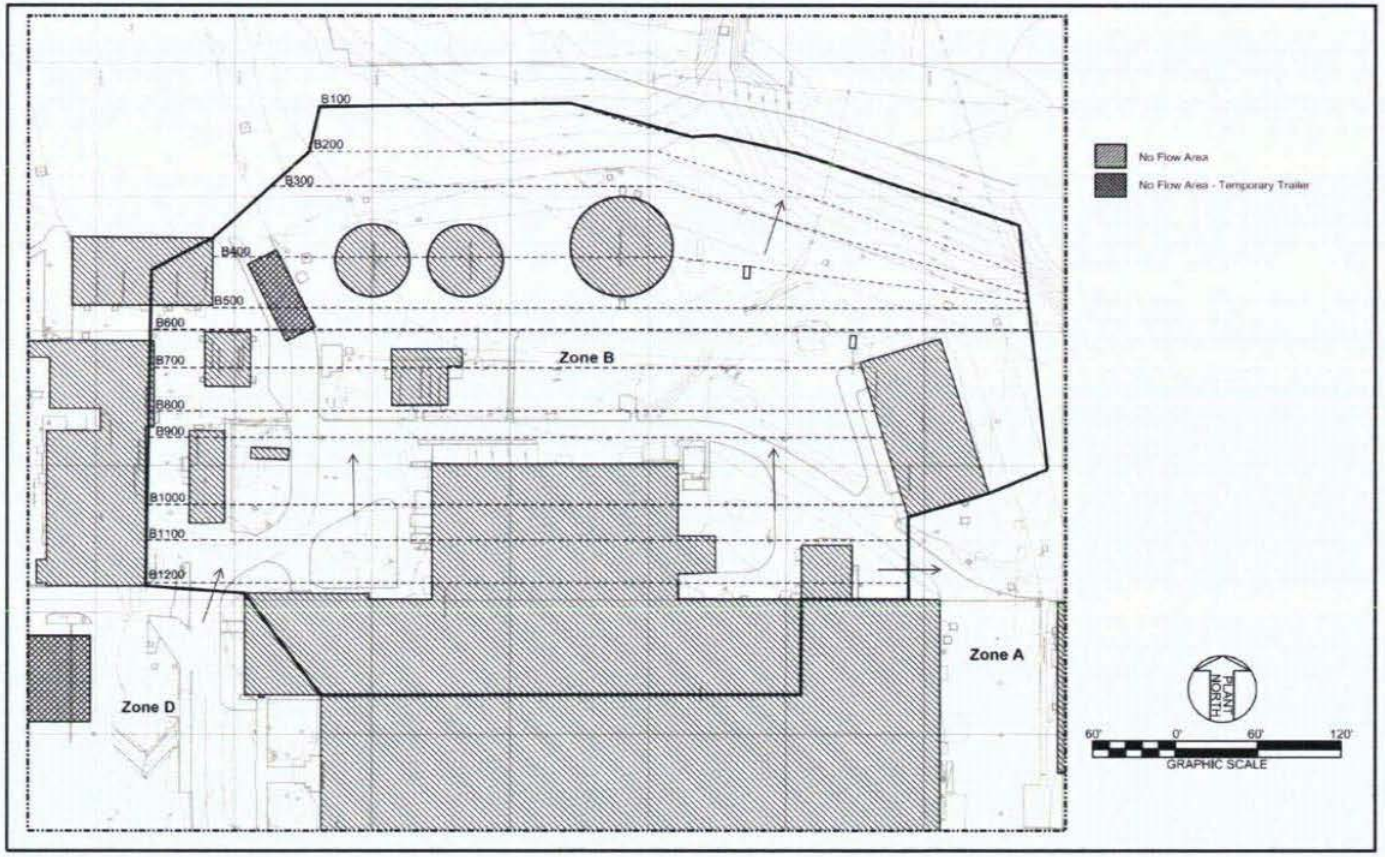


Figure 2.1-6: Cross Sections for LIP Evaluation - Zone C

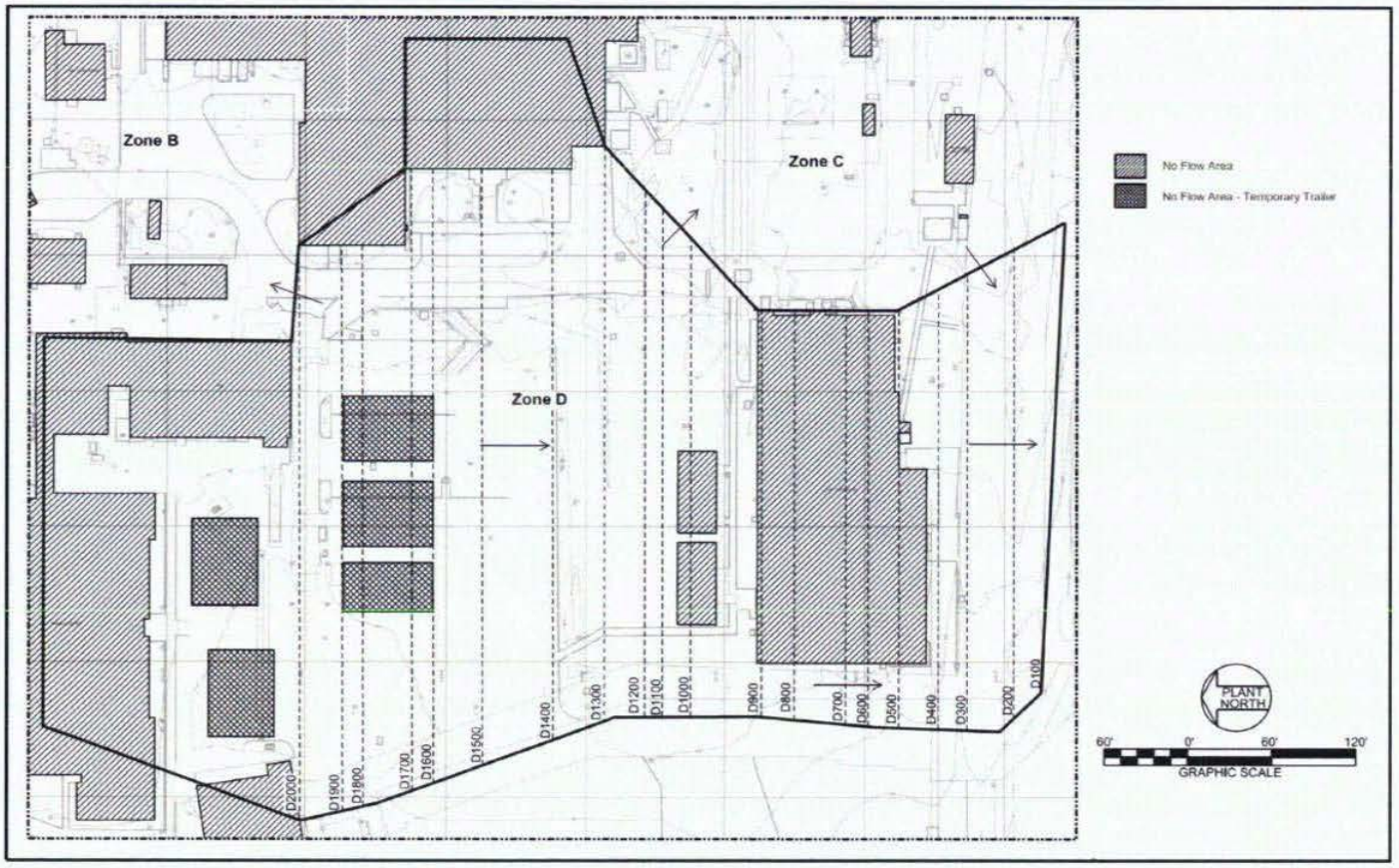


Figure 2.1-7: Cross Sections for LIP Evaluation - Zone D

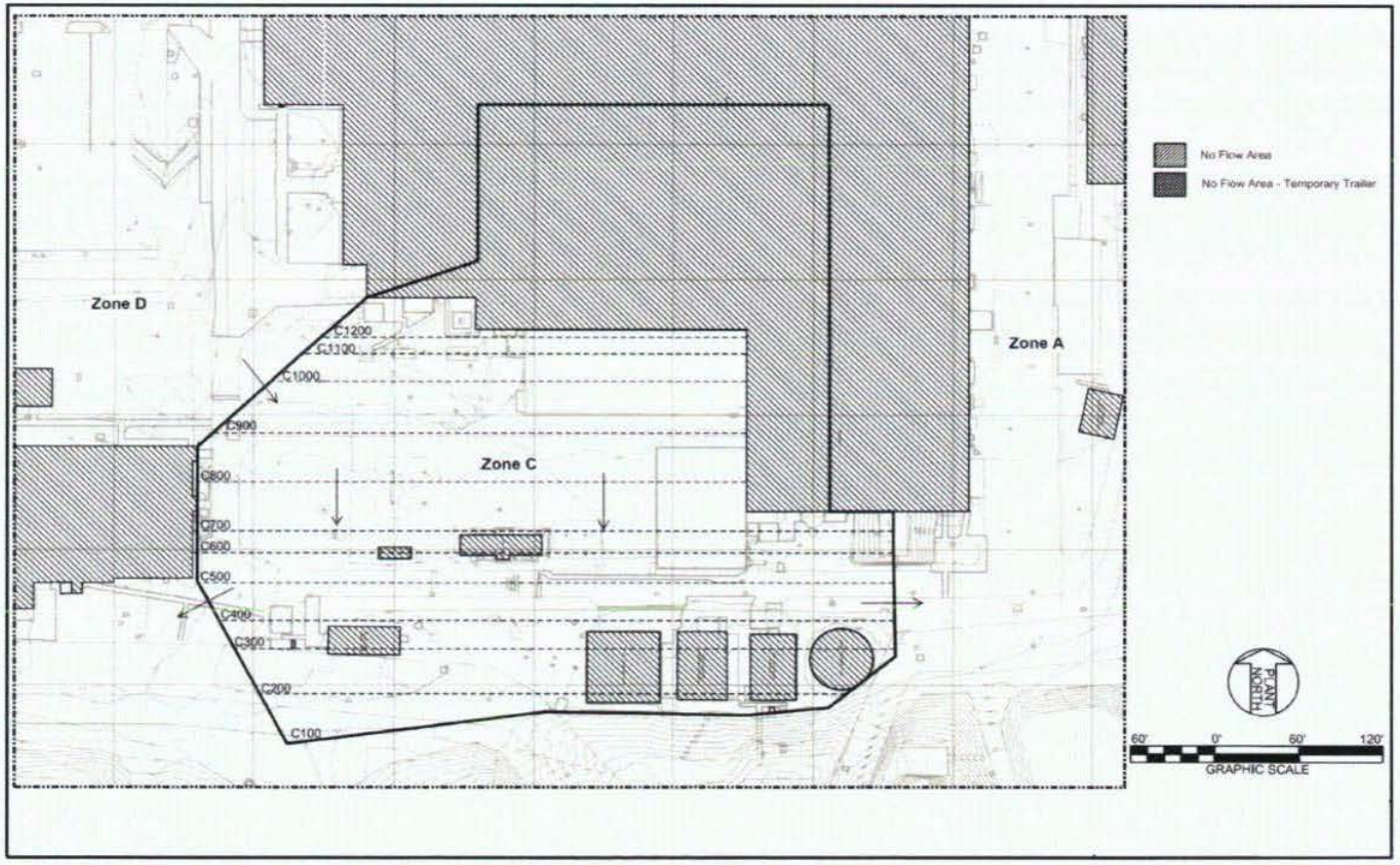


Figure 2.1-8: HEC-RAS Cross Sections for Zone A

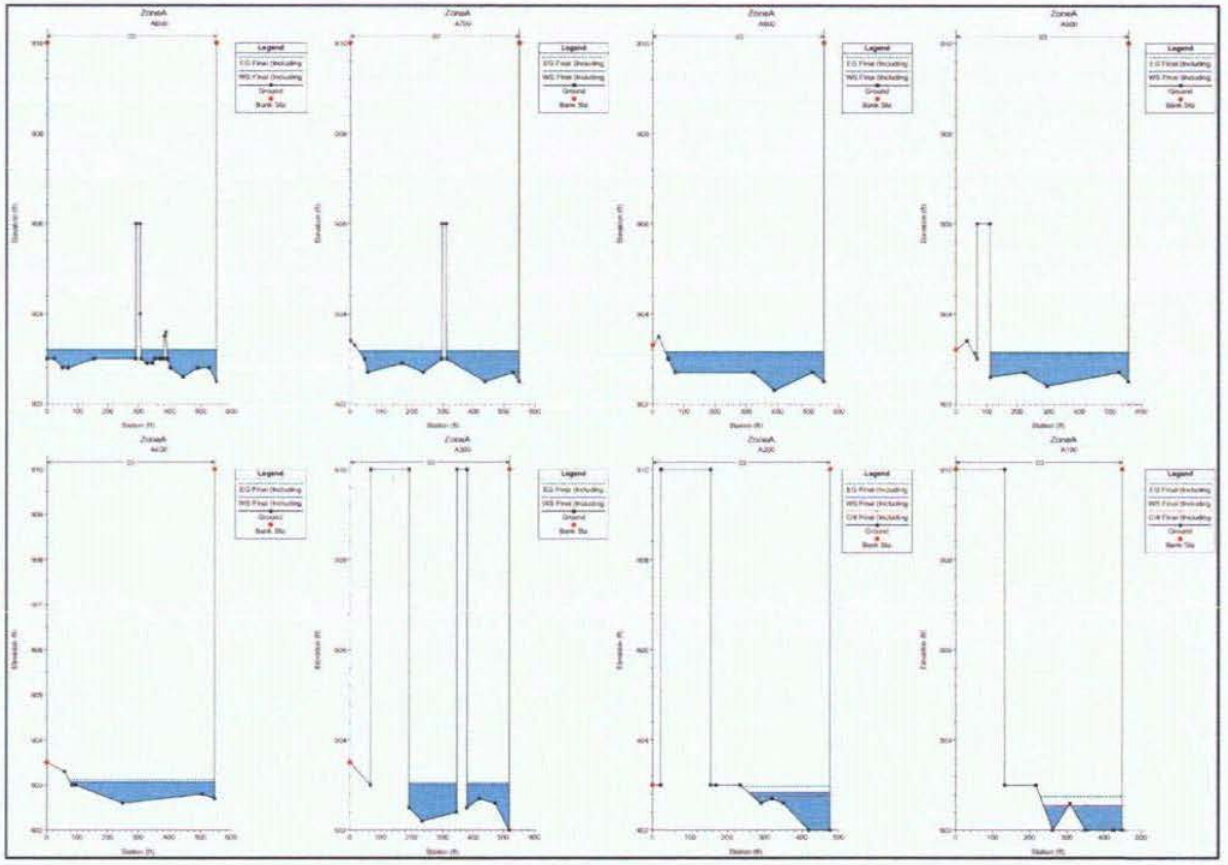


Figure 2.1-9: HEC-RAS Cross Sections for Zone B (sheet 1 of 2)

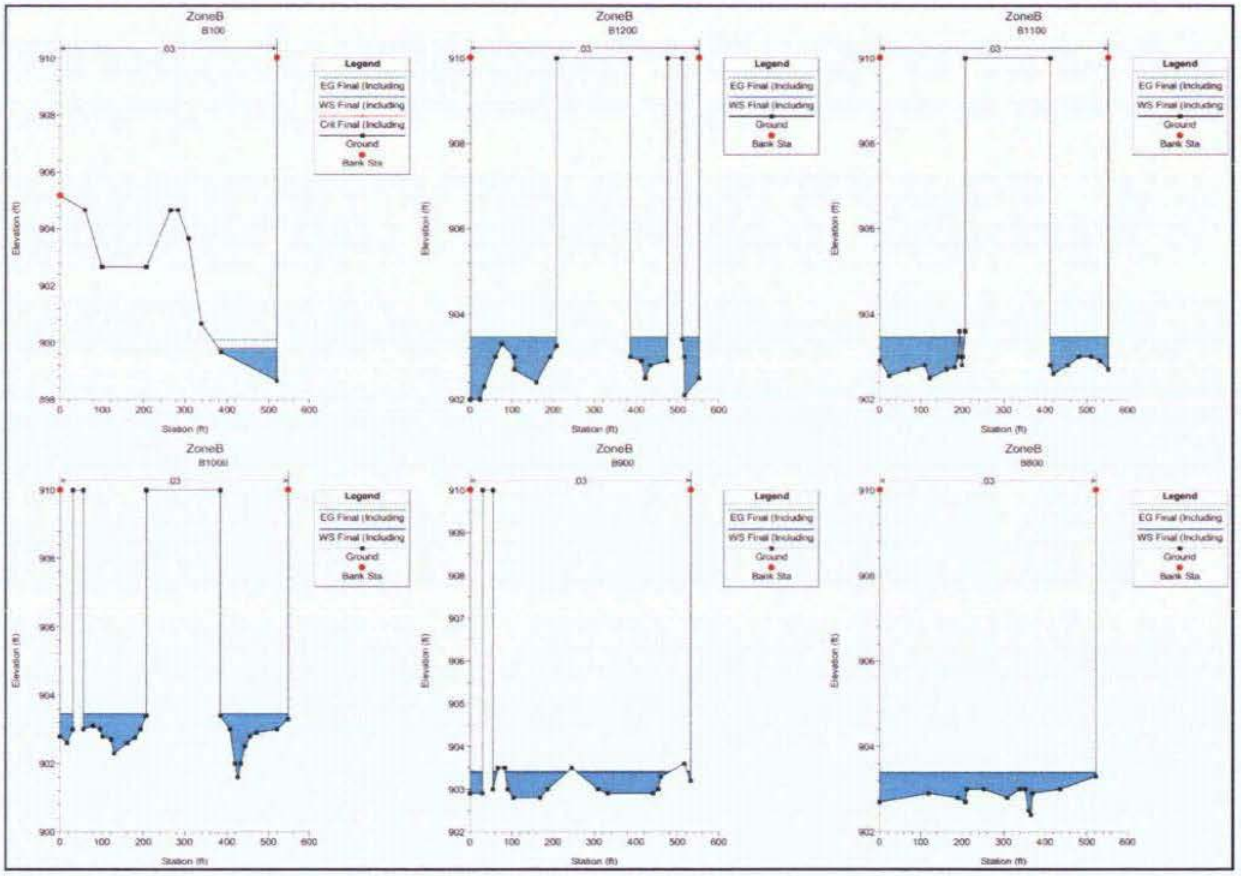


Figure 2.1-9: HEC-RAS Cross Sections for Zone B (sheet 2 of 2)

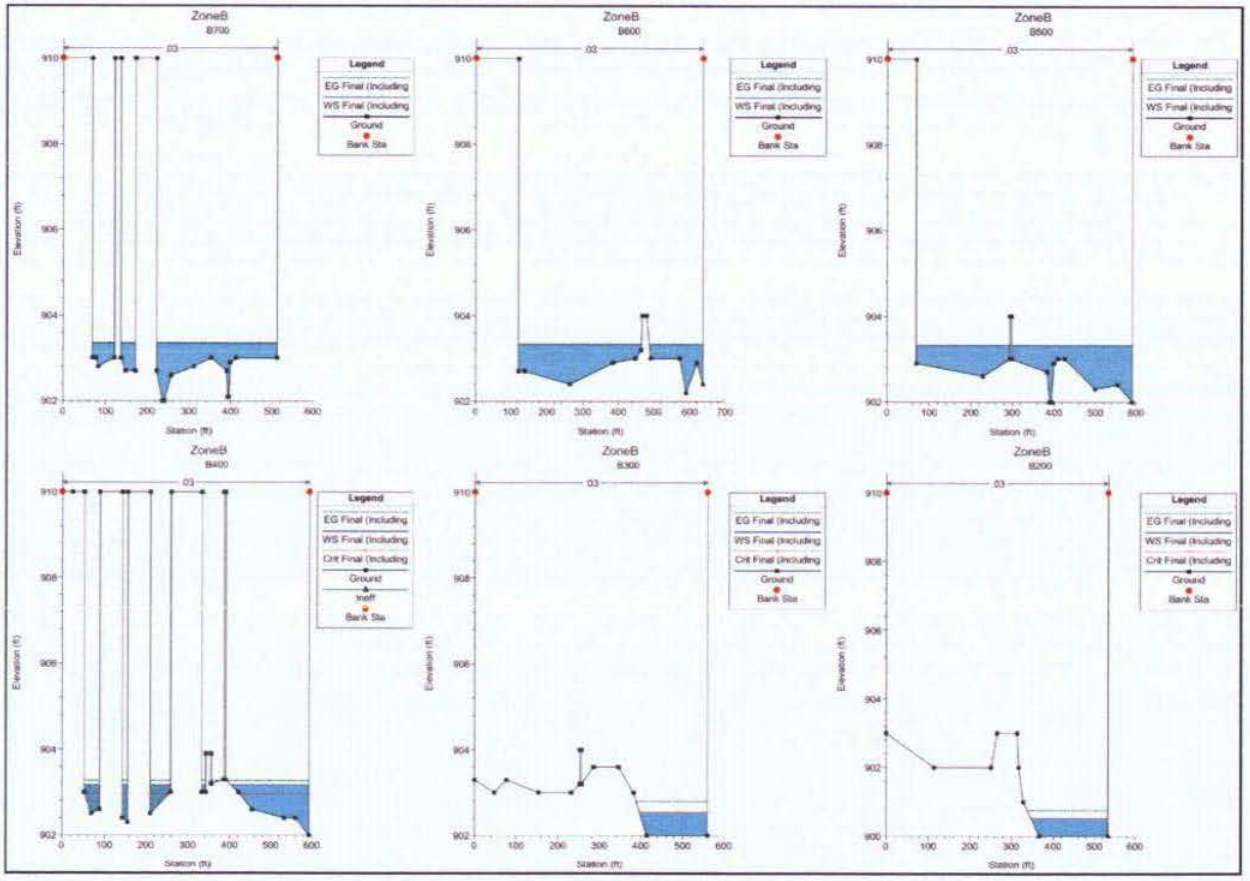


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 1 of 6)

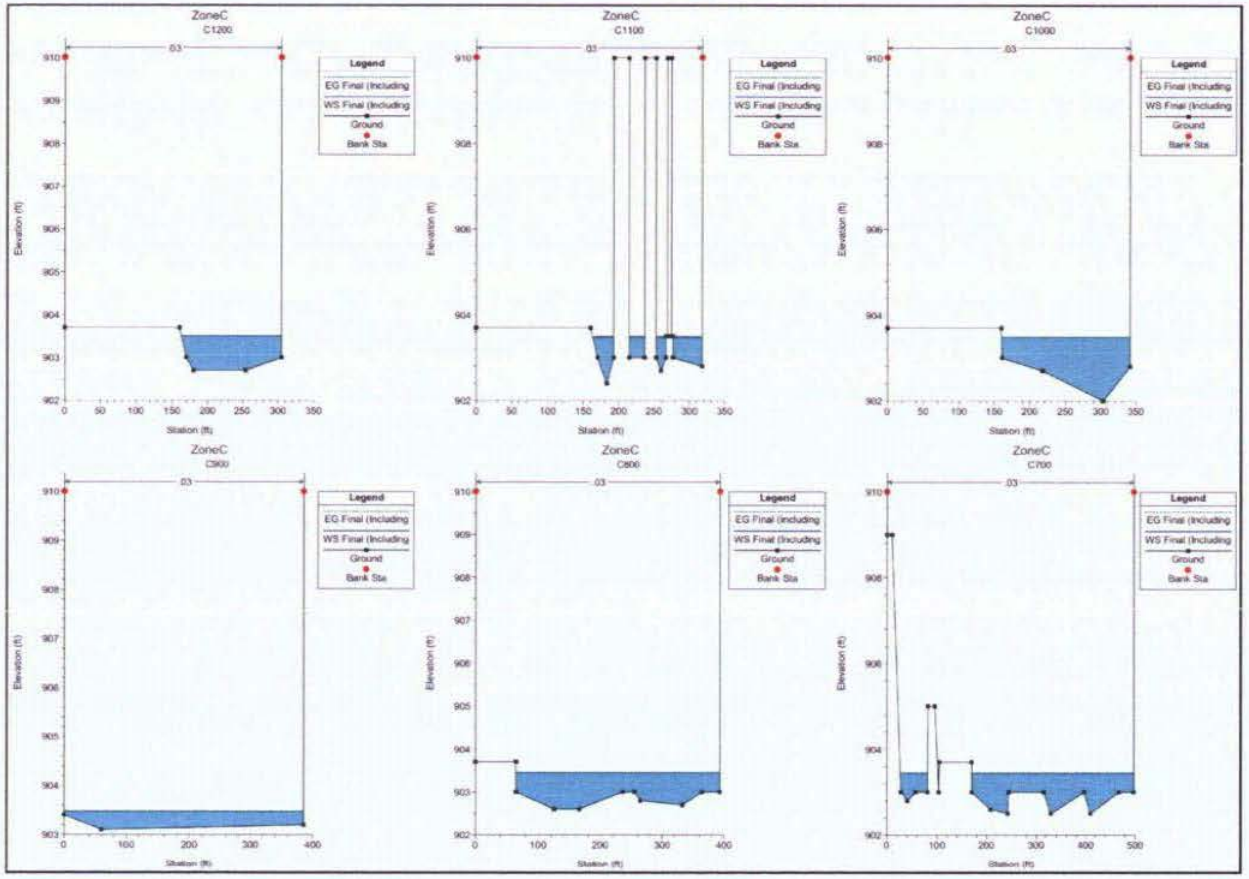


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 2 of 6)

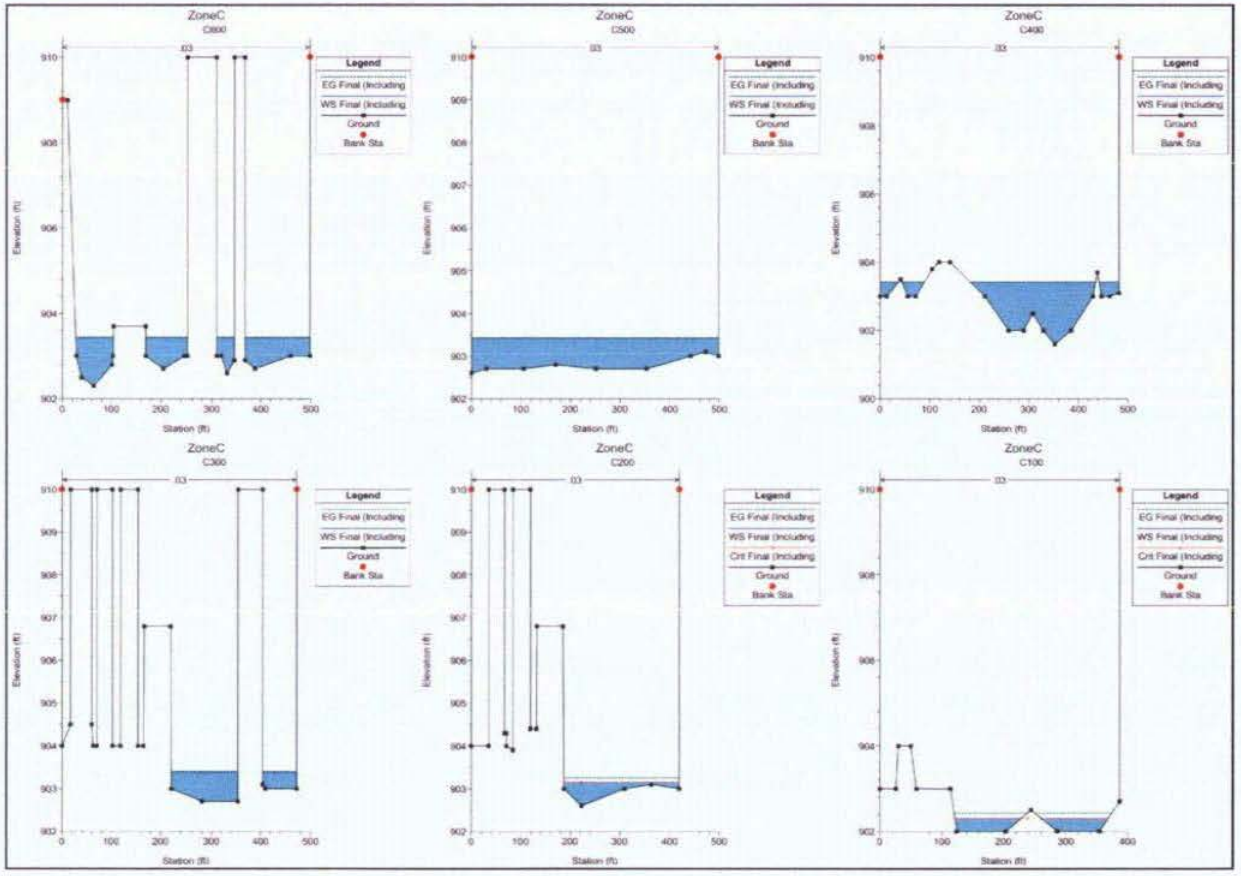


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 3 of 6)

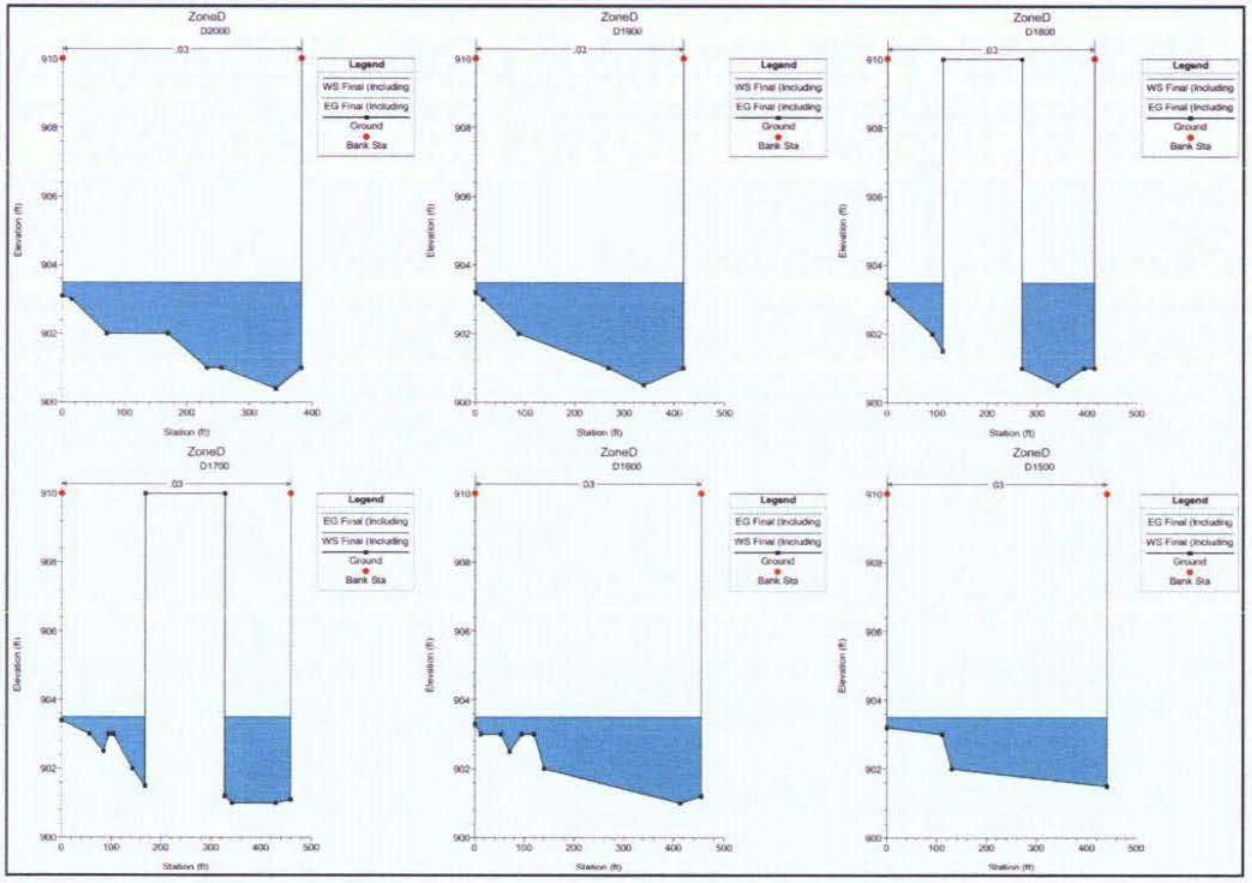


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 4 of 6)

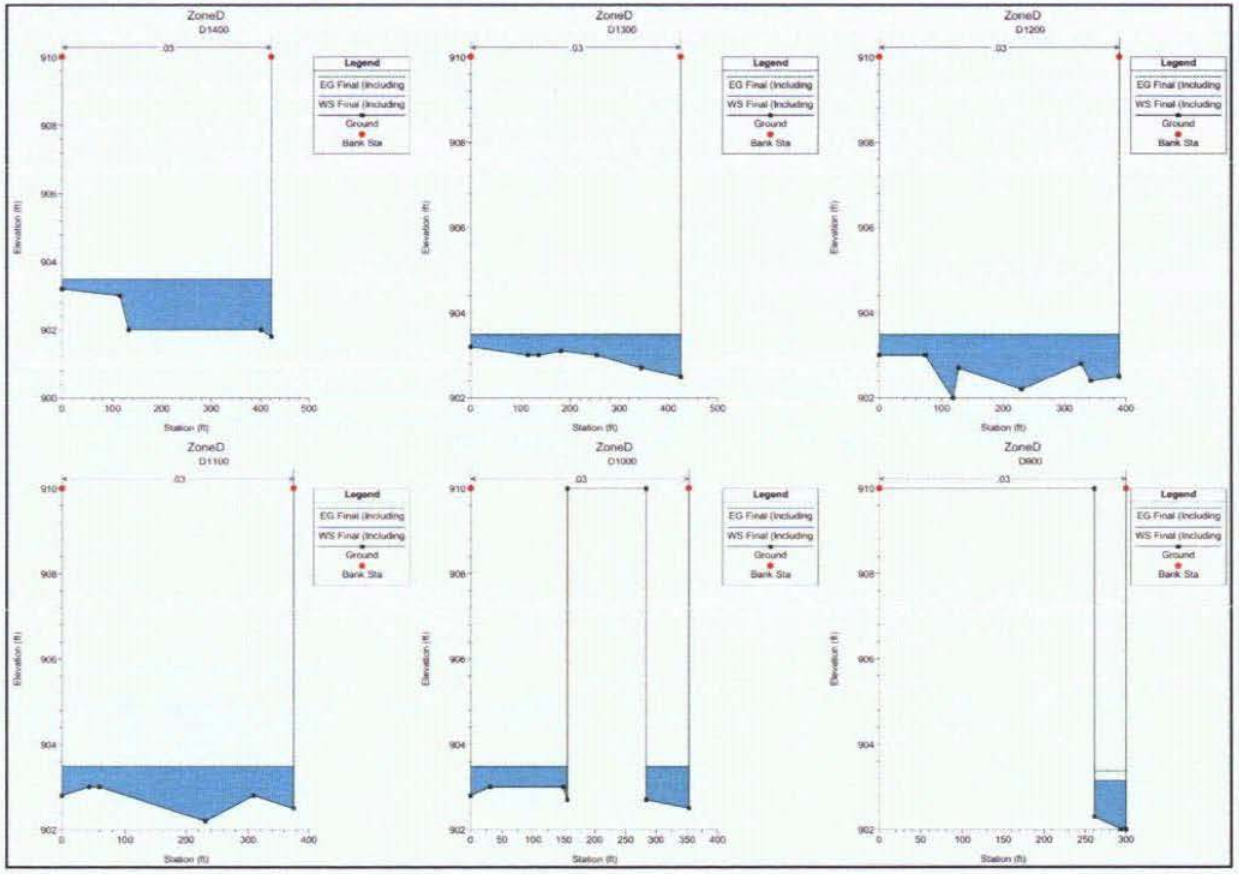


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 5 of 6)

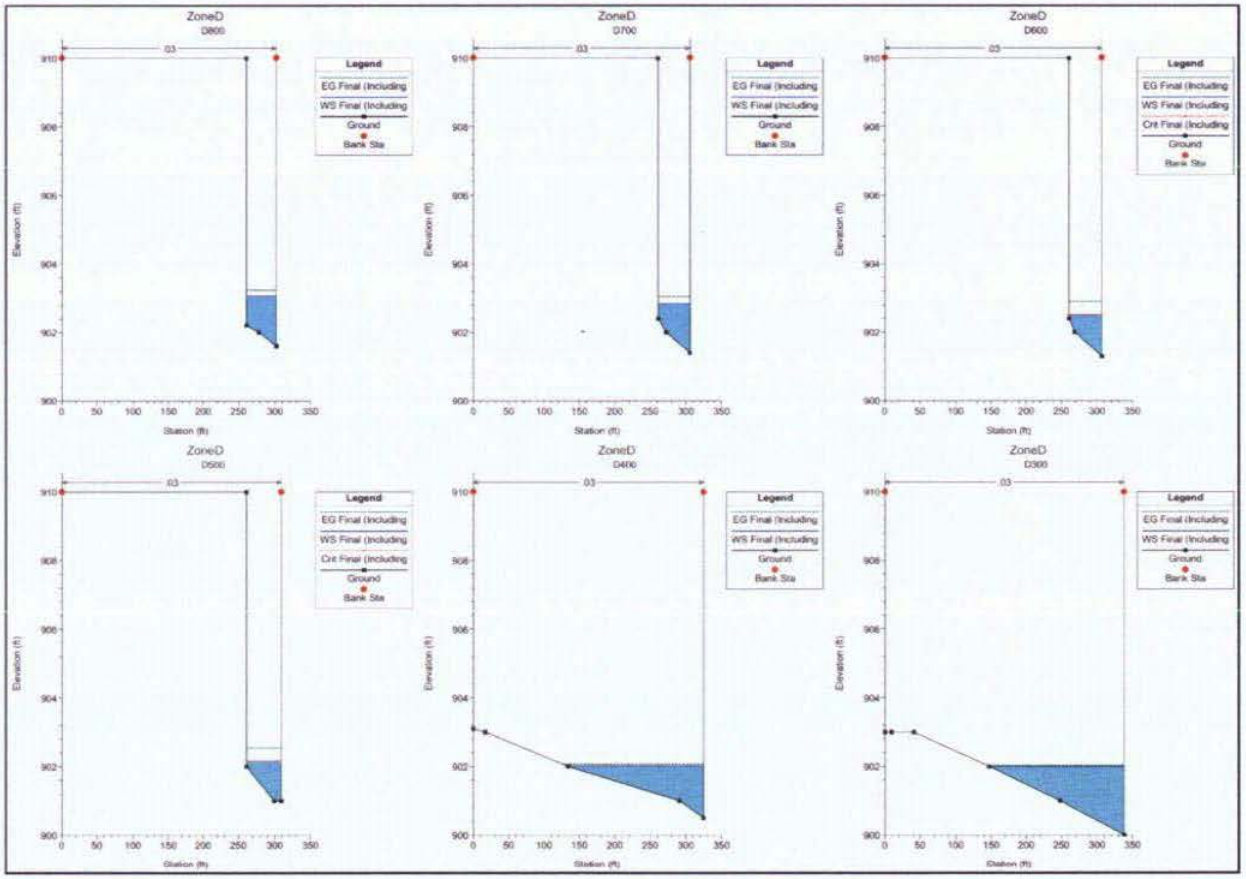
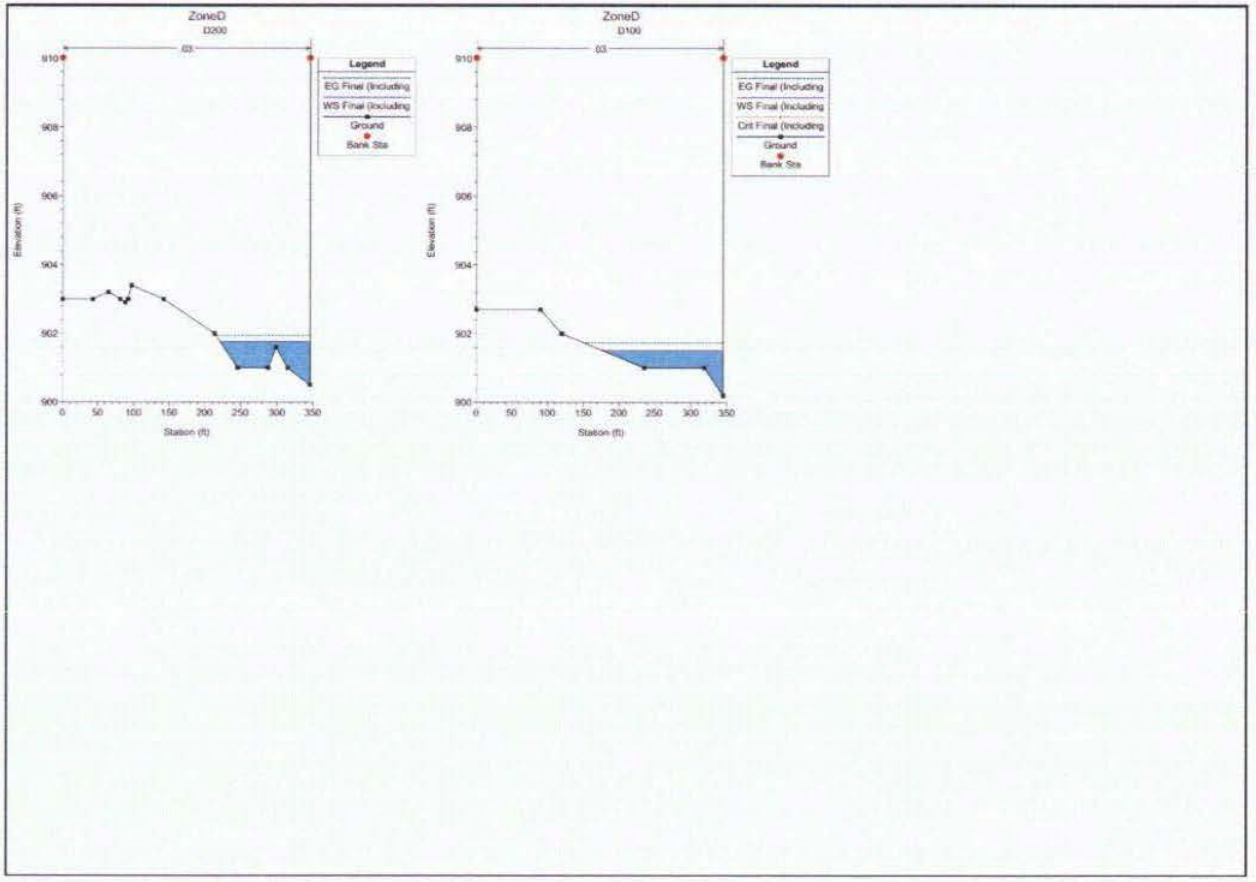


Figure 2.1-10: HEC-RAS Cross Sections for Zone C (sheet 6 of 6)



2.2 FLOODING IN STREAMS AND RIVERS (PMF)

The Cooper Nuclear station (CNS) is located on the west (right) bank of the Missouri River in Nemaha County near Brownville, Nebraska at approximate Missouri River Mile (RM) 532.5 (see Figure 2.2-1).

In this section of the report, the Probable Maximum Flood (PMF) in the Missouri River was evaluated to assess the flooding hazard on the safety-related facilities at CNS. The PMF is the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of the Probable Maximum Precipitation (PMP) and other hydrologic factors favorable for maximum flood runoff, such as sequential storms and snowmelt (Reference 2.2-1).

The hydrologic conditions in the Missouri River at CNS are complex due to size of the contributing Missouri River watershed (414,900 square miles above Rulo, Nebraska near CNS, United States Geological Survey [USGS] gage number 06813500 [Reference 2.2-2]) and the six System dams (Fort Peck, Garrison, Oahe, Big Bend, Fort Randall, and Gavins Point) located on the mainstem of the Missouri River upstream of CNS (Figure 2.2-2 sheet a). The Missouri River is the longest river in the United States, draining one-sixth of the country (Reference 2.2-3). The U.S. Army Corps of Engineers (USACE) regulates all of the System dams. They are regulated as a hydraulically and electrically integrated system. Runoff from upstream of the System dams is stored in the six reservoirs. USACE controls the water release from the System dams. Released water from the most downstream dam in the System, Gavins Point Dam, flows down the Missouri River toward CNS. In addition, there are also a large number of other dams (referred to as "Non-System" dams) in the drainage basins contributing to the Missouri River downstream of the "System" dams and upstream of CNS (see Figure 2.3-1).

The U.S. Nuclear Regulatory Commission (NRC), in Section 5.3 of NUREG/CR-7046 (Reference 2.2-1), suggests use of flood simulation models developed by federal agencies, such as USACE. Following such recommendations, the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS, Reference 2.2-13) and the Hydrologic Engineering Center River Analysis System (HEC-RAS, Reference 2.2-5) were used to perform the hydrologic (Section 2.2.2) and hydraulic (Section 2.2.3) analyses, respectively.

The PMF value for CNS was developed using the following steps:

- Determine probable maximum precipitation (PMP) over the basins that drain into the Missouri River below Gavins Point Dam and upstream of CNS.
- Develop the PMF hydrographs by applying the PMP over the basin.
- Route the PMP runoff using one-dimensional (1-D) unsteady basin-scale hydraulic model and generate boundary conditions for a reach-scale two-dimensional (2-D) hydraulic model.
- Determine depth and velocity of the flow at CNS during PMF using a reach-scale 2-D hydraulic model.

- Calculate wind setup, wind-wave runup, and hydrostatic and hydrodynamic forces on systems, structures, and components (SSCs) and other buildings at CNS due to the PMF.
- Calculate debris impact forces and evaluate the erosion and sedimentation at the site.

Details of each step of the analysis are provided in the following sections.

2.2.1 Probable Maximum Precipitation (PMP)

In this section of the report, estimation of a realistic, yet conservative, PMP over the Platte River Basin and the adjoining reaches of the Missouri River identified as the Lower Basin (Fort Calhoun) and the Lower Basin (Cooper) is discussed (see Figure 2.2-3). This PMP was used to determine the maximum water level at CNS at Missouri RM 532.5 resulting from the PMF in the Missouri River.

2.2.1.1 Basin Delineation

The applicable basin was delineated using the United States Department of Agriculture (USDA) Watershed Boundary Dataset (Reference 2.2-6). Watershed boundaries define the aerial extent of surface water draining to a particular point, which is CNS. The watershed boundaries were defined through the use of hydrologic units (HUs) to establish a baseline rain gage boundary framework, accounting for all land and surface areas. Geographical Information System (GIS) software, specifically Environmental Systems Research Institute (ESRI) ArcGIS (Reference 2.2-7), was used to delineate the basin. The delineated basin outline is shown in Figure 2.2-3.

2.2.1.2 PMP Alternatives

Section 9.2.1.1 of ANSI/ANS-2.8-1992 (Reference 2.2-8) and Appendix H of NUREG/CR-7046 (Reference 2.2-1) specify three different alternatives for a flood reevaluation analysis. The alternatives comprise a combination of:

- ***Alternative 1*** - mean monthly (base) flow, median soil moisture, antecedent or subsequent rain (the lesser of rainfall equal to 40% of the PMP and a 500-year rainfall), PMP, and waves induced by the 2-year wind speed applied along the critical direction.
- ***Alternative 2*** - mean monthly (base) flow, probable maximum snowpack, a 100-year, snow-season rainfall, and waves induced by the 2-year wind speed applied along the critical direction.
- ***Alternative 3*** - mean monthly (base) flow, a 100-year snowpack, snow-season PMP, and waves induced by the 2-year wind speed applied along the critical direction.

An evaluation of these alternatives showed that Alternatives 2 and 3 do not create an extreme peak in a short time period as Alternative 1 does. Therefore, Alternative 1 was used in this analysis as it was the most conservative approach.

The 500-year, 72-hour rainfall value for a point at the centroid of the PMP analyses for the basin was obtained from National Oceanic and Atmospheric Administration (NOAA) Atlas 14 (Reference 2.2-9).

Analysis showed that applying a 40% PMP over the basin results in approximately one half the rainfall volume that a 72-hour 500-year storm event would create. Therefore, the 40% PMP was considered as the antecedent rain. The 40% PMP storm event will precede the PMP storm event by 3 days (3 to 5 days allowable in ANSI/ANS-2.8-1992, Reference 2.2-8) as this period of transition is meteorologically reasonable for this region of the United States.

2.2.1.3 PMP Development

PMP storm depth, spatial distribution, centering, and orientation pattern for a 72-hour storm period adopted for the drainage basins upstream of CNS and downstream of Gavins Point Dam were derived following the procedures described in the National Weather Service (NWS) Hydrometeorological Reports 51 and 52 (HMRs 51 and 52, References 2.2-10 and 2.2-11, respectively). The PMP estimates obtained from these HMR procedures are location-specific and have accounted for orographic and seasonal effects.

The lower portion of the Basin (38,672 square miles), including the Platte River watershed below Lake McConaughy and the portion of the Missouri River watershed between Gavins Point Dam and CNS, was chosen as the watershed area of interest for this PMP analysis. Centering the PMP event over the lower portion of the Basin produces the most hydrologically conservative result. ANSI/ANS-2.8-1992 (Reference 2.2-8) recommends that enough storm center positions be considered to ensure that the most critical condition has been determined. In this study, four storm centers (centroids) and orientations (Figure 2.2-4) were used so that the hydrologic sensitivity and resultant flows from the analyses of the PMP could identify through hydrologic modeling (Section 2.2.2) which location would produce the highest water elevation at CNS. The following steps were performed in establishing the PMP resulting in the highest water elevation at CNS:

- Obtain the all-season precipitation values from HMR 51 at all four storm centers for basins ranging from 10 to 20,000 square miles for the 6-, 12-, 24-, 48-, and 72-hour durations.
- Because the maximum basin size available from HMR 51 data is 20,000 square miles, only four basin sizes less than the 20,000 square miles were analyzed, in order to develop an envelope that generates the maximum precipitation volume. Basin sizes of 20,000-, 15,000-, 10,000-, and 6,500-square miles were chosen to develop the envelopment curves.
- The incremental differences between 0 to 6-hour, 6 to 12-hour, and 12 to 18-hour precipitation values were then calculated and the digitized isohyet pattern was centered on the basin centers and oriented along the long axis of each basin (Figure 2.2-4) using GIS software.
- Determine the cumulative rainfall volume for each of the four enveloping basin sizes to determine which size has the highest volume and, therefore, the highest PMP value. The 20,000 square miles of watershed yielded the highest precipitation volume with the exception of the fourth center (P4 in Figure 2.2-4) analysis in which the 15,000-square mile watershed yielded the highest precipitation volume.

- Distribute the storm-area-averaged PMP over the drainage basin and develop precipitation depths for each storm area and 6-hour temporal distribution period (see Table 2.2-1). The 40% PMP storm event was then determined by multiplying the data in Table 2.2-1 by 0.4.

2.2.2 PMP Runoff Hydrographs

A hydrologic model of the Missouri River extending from downstream of Gavins Point Dam to Kansas City, Missouri was originally developed by USACE Omaha District (USACE-OD, Reference 2.2-12) using HEC-HMS Version 3.5. This existing model was obtained and modified for this study to generate hydrographs from the basins upstream of CNS and downstream of Gavins Point Dam that contribute to the Missouri River PMF at CNS. The process used to develop the PMF hydrographs for the watersheds upstream of CNS includes the following:

- Review basin model configurations and parameterizations.
- Validate the reasonableness of the model at key locations for each basin.
- Determine the highest-computed PMF hydrograph at CNS based on hydrologic modeling.

2.2.2.1 Hydrologic Model Development

The eight HEC-HMS models (one for each river basin depicted in Figure 2.2-2) received from USACE-OD used the following methods: Deficit and Constant loss, ModClark transform, Recession baseflow, and Muskingum-Cunge channel routing. A summary of model input parameter definitions and references are provided in Table 2.2-2. The analysis incorporated the Hierarchical Hazard Assessment (HHA) approach outlined in Section 2 of NUREG/CR-7046 (Reference 2.2-1). Consistent with the HHA approach, any adjustments to model input parameters were made globally conservative to the extent possible without increasing the level of detail of the model.

The USACE-OD basin model inputs were reviewed and verified by comparing the values with the corresponding range of values available in standard literature. The spatial distribution of the initial deficit, constant loss rate, and maximum deficit values were evaluated by creating a map of the basin and examining the variation in values across the basin model as compared to hydrologic soil group. In addition, the loss and transform parameters were verified for a typical subbasin within each basin or major tributary using values and equations found in the standard literature.

The United States Geological Survey (USGS) indicates that a small portion of each drainage area is non-contributing area; however, conservatively all areas were considered as contributing in the model. The PMF is such an extreme event that normal drainage patterns may be altered; therefore, inclusion of the non-contributing drainage area would produce a conservative estimate of the PMF hydrograph at CNS.

2.2.2.2 Hydrologic Model Validation

Daily stream flow data were used for upstream boundary conditions to validate model performance at key locations within the basin. Such data were obtained from the USGS and Nebraska Department of Natural Resources (NDNR) Websites (References 2.2-2 and 2.2-12, respectively).

To examine the reasonableness of the magnitude and timing of the peak flow and the volume of the hydrographs, the models were executed for the largest historic storm for which data were available and computed hydrographs were compared to stream gage values at the key locations within the basin. The selected storm was a 1984 storm for all basins except the Platte River basin, which used a 2010 storm. The model hydrographs were compared to measured hydrographs by computing the Nash-Sutcliffe model efficiency coefficient (NSE). The NSE generally is used to assess the predictive power of a model's ability to reproduce observed data. NSE ranges between negative infinity and 1.0 (1 inclusive), with NSE = 1 being the optimal value. Essentially, the closer the model efficiency is to 1, the more accurate the model. In addition to NSE, the Pearson correlation coefficient (r) was determined. The coefficient provides a measure of the linear relationship between two variables, in this case, measured and computed discharge. The models were considered validated when the modeled versus measured hydrograph NSE was greater than 0 and/or r was greater than 0.7, unless otherwise justified.

If required, model parameters were adjusted to obtain an acceptable fit to USGS hydrographs. Analysis showed that the basin response was most sensitive to the constant loss rate. Therefore, the constant loss rate was reduced in all models and other parameters were adjusted until a reasonable match to gage records was realized. An attempt was made to globally reduce all constant loss rates in the basin; however, it was found that reducing the constant loss rate variably within the basin provided the best fit for each gage location. Some of the reduced constant loss rates were lower than the recommended range listed in Table 11 of HEC-HMS Technical Reference Manual (Reference 2.2-13), but the modified rates are considered acceptable and produced more conservative model results. To match the baseflow and recession limb of the hydrograph, the recession constant and initial discharges were further adjusted. The computed hydrographs provided a relatively good fit to the measured values in terms of peak discharge, volume, and timing, especially for the magnitude of the historic storm used and the size and relative coarseness of the model.

2.2.2.3 PMF Hydrograph Determination

The PMF hydrograph is a result of applying a PMP over a specific area and routing the resulting hydrographs to a point of interest, which is CNS. PMP positions (as discussed in Section 2.2.1, Figure 2.2-4) were executed using the validated model for the watersheds upstream of CNS and downstream of Gavins Point Dam. As discussed in Section 2.2.1.2, the 40% PMP value was considered as the antecedent or subsequent storm. According to ANSI/ANS-2.8-1992 (Reference 2.2-8), a preceding storm usually is the more critical, but an alternative subsequent timing should also be investigated. Therefore, in addition to the antecedent storm with the PMP, another simulation was performed with the PMP and the subsequent storm. This was done only for the position that resulted in the highest PMF at CNS to compare whether the antecedent or subsequent condition resulted in a higher peak. The sequential timing (3-day lag) was the same as the antecedent condition.

NUREG/CR-7046 (Reference 2.2-1), recommends increasing the unit hydrograph peak discharge by 5% to 20% and reducing the unit hydrograph lag time by 33% to account for non-linearity effects during an extreme event such as the PMF. The HEC-HMS modeling framework in this analysis used the ModClark transform, which does not produce a unit hydrograph. Therefore, instead of increasing the peak discharge or reducing the lag time of a unit hydrograph, the time of concentration in all subbasins was reduced by 33% as a surrogate for reducing unit hydrograph lag time and the constant loss rate was reduced so that the peak discharge was increased from 5% to 20%.

ANSI/ANS-2.8-1992 (Reference 2.2-8) recommends mean monthly flow during the month of occurrence of the PMF to be used as the baseflow at the beginning of an antecedent storm. Where USGS gage data were available, the initial baseflow values of the subbasins draining to the gage locations were adjusted to match the maximum mean monthly flow during the months when the PMP will most likely occur. The initial baseflow values in subbasins for which no USGS gage data were available were not changed. The four PMP positions in this Midwestern part of the country will likely occur in the warmer months of the year. Typically, the highest mean monthly flows at the USGS gages used for this study occur in May, June, and July.

PMF hydrographs at Brownville, Nebraska for all four PMP positions with 40% antecedent storm are presented in Figure 2.2-5. Similarly, hydrographs with 40% subsequent PMP are shown in Figure 2.2-6. Analysis results (Figure 2.2-5 and Figure 2.2-6) showed that the 40% PMP Position 3 (Figure 2.2-2, sheet c) followed by 3 days of no precipitation, followed by the PMP Position 3 (Figure 2.2-2, sheet c) resulted in the highest peak flow at CNS, which was considered as the controlling PMF hydrograph and was routed through the Missouri River using dynamic hydraulic modeling (as discussed in Section 2.2.3).

Position 1, centered over the Platte Basin and Loup River Basin (Figure 2.2-2, sheet a), resulted in the lowest peak PMF at CNS. This is due to the presence of sandy soils in the Loup Basin and the associated relative high constant loss rates compared to adjacent basins. Position 4 (Figure 2.2-2, sheet d), centered near CNS, resulted in the second lowest peak PMF at CNS. This is due to the storm being centered over the lower portion of the contributing basin at CNS, and thus not having the runoff volume necessary to produce a relatively large PMF. Position 2 (Figure 2.2-2, sheet b) generated the second highest peak PMF at CNS. The Platte River and Elkhorn River contributed most of the peak discharge.

Figure 2.2-5 shows that the PMP Position 3 hydrographs have a peak discharge of approximately 832,250 cubic feet per second (cfs) at CNS. Figure 2.2-7 shows the location and RM of each junction that their outflow hydrographs from the hydrologic model were used as an input to the hydraulic simulation described in Section 2.2.3.

2.2.3 Water Level Determinations

The hydraulic conditions in the Missouri River between Gavins Point Dam (approximate RM 810) and CNS during flood flows are complex due to the presence of revetment, river training structures, levees, roadway embankments, river meanders, and bridges. To evaluate this complexity, a multi-leveled modeling approach was adopted in this flood hazard reevaluation study. A 310-mile basin-scale one-

dimensional (1-D) HEC-RAS unsteady model was used to predict the PMP hydrograph (as discussed in detail in Section 2.2.2) translation and attenuation in the Missouri River from Gavins Point Dam to CNS. The results from the 1-D basin-scale routing model were then used as upstream boundary conditions for a reach-scale two-dimensional (2-D) hydraulic model. This reach-scale model, approximately 46 miles in length, was further used to predict the complicated interaction between the river channel and overbank areas that provides an estimate of flow distributions near CNS. The results from this 2-D reach-scale model were used to predict water surface elevations and velocities at CNS for the PMF.

2.2.3.1 One-Dimensional Steady-State Hydraulic Model

In 2003, an unsteady model was developed and calibrated by the USACE-OD for the Upper Mississippi River System Flow Frequency Study (UMRSFFS), Reference 2.2-14. The Federal Emergency Management Agency (FEMA) regulatory model was developed in HEC-RAS by converting the UMRSFFS unsteady model to a steady-state model and then calibrated to the unsteady model 100-year flood event results (Reference 2.2-15). This model was obtained from FEMA and used for evaluation of PMF at CNS. The model had undergone rigorous review prior to acceptance by FEMA. The regulatory FEMA HEC-RAS steady-state model of the Missouri River was used to develop a 1-D HEC-RAS steady-state hydraulic model of the Missouri River for CNS PMF analysis that became the basis for the 1-D HEC-RAS unsteady model of the Missouri River (as discussed in Section 2.2.3.2). The process used to develop the 1-D steady-state hydraulic model from the regulatory FEMA model included:

- The hydraulic model cross sections were modified and updated with post 2011 flood bathymetric and Light Detection and Ranging (LiDAR) survey data.
- Manning's roughness coefficients varied both horizontally and from cross section to cross section, but remained within the range of published values. In certain instances, Manning's roughness coefficients outside of the published ranges for overbank land uses were used to describe flow resistance from buildings or other overbank structures.
- The sensitivity of steady-state hydraulic results at the Missouri River near CNS (RM 532.35) was assessed with a discharge of 290,000 cfs by increasing and decreasing the Manning's roughness coefficients in the model domain by 10% and 20%. Analysis demonstrated that the model results were sensitive to Manning's roughness coefficients, which is consistent with guidance in NUREG/CR-7046, Section 5.4 (Reference 2.2-1). However, the majority of stage-discharge comparisons from this sensitivity analysis are within the 95% prediction interval.
- The sensitivity of steady-state hydraulic results for the Missouri River near CNS (RM 532.35) was assessed with a discharge of 290,000 cfs by increasing and decreasing the normal depth slope of the downstream boundary from 0.0001 to 0.0002 and 0.00005. The mean channel velocity and water surface elevation WSEL were identical between the three runs at this location. This indicates that the downstream boundary condition is located at an adequate distance downstream of CNS.
- The model was calibrated for 100-, 200-, 500-year, and 2011 flood events (Reference 2.2-14) to better replicate published stage-discharge relationships. The NRC, in NUREG/CR-7046, Section 5.5 (Reference 2.1-1), prescribes validation of the

simulation models using the largest historical flood near the site. The 2011 flood event on the Missouri River resulted in the highest peaks within the modeled reach since 1952 (i.e., prior to construction of the five lower mainstem dams). Therefore, the measured data at USGS stream gages were considered the best available information because the data were collected during the 2011 flood event. The computed WSELs compared favorably to measured gage data and published flood insurance study (FIS) values for the 100-, 200-, and 500-year discharges within the modeled reach. In addition, the computed WSELs compared reasonably to measured values within the reach for the 2011 flood flows. Therefore, the model was considered calibrated for flows between the 100- and 500-year events (including the 2011 flood event) and, in accordance with NUREG/CR-7046, Section 5.5 (Reference 2.1-1), provides some assurance that the estimated design-basis floods will not be underestimated. In the calibrated HEC-RAS steady-state model, the channel Manning's roughness coefficients varied from 0.025 to 0.029 in the channel and generally from 0.020 to 0.120 in overbank areas.

2.2.3.2 One-Dimensional Unsteady Hydraulic Model

The 1-D steady-state hydraulic model of the Missouri River, as discussed in the previous section, was the primary data source for the HEC-RAS unsteady model. With the exception of modifications to the Manning's roughness coefficients, levee locations, and ineffective flow areas, the geometry from the steady-state model was used in the unsteady flow analysis. Figure 2.2-1 shows the extent of the modeled reach. The model was used to generate boundary conditions, including an upstream inflow hydrograph, upstream flow distributions, and a downstream stage hydrograph for a reach-scale unsteady 2-D model of the Missouri River valley in the proximity of CNS. Specifically, this model predicted translation and attenuation of hydrographs that contribute to the PMF at CNS, considering inflows at Gavins Point Dam and tributaries throughout the modeled reach. The process used to develop the unsteady model included:

- Validate the HEC-RAS unsteady model with input and gage data from the 2011 flood event, making the necessary changes to replicate observed discharges, stages, and timing.
- Analyze PMF flow scenarios using the HEC-RAS unsteady model.
- Establish inflow discharge hydrographs at RM 556.16 and outflow rating curve(s) at RM 510.03 to be used as boundary conditions for the unsteady 2-D model.

2.2.3.2.1 *HEC-RAS Unsteady Model Development*

There are approximately 133.5 total miles of federal levees along the Missouri River. To address the uncertainty associated with predicting event routing through the modeled reach, some simplifying assumptions were made in order to develop a conservative estimate of flood translation and attenuation. Only federal levees were included in the model, which is consistent with the approach used by USACE in the UMRSFFS (Reference 2.2-14). It is difficult to predict when, where, and how levee overtopping or breaching will occur over approximately 133.5 miles of levee using a 1-D unsteady model. Overtopping of the levee can be modeled considering levees as lateral weirs, but the assumption was made that the inclusion of lateral weirs was unnecessary to calculate a conservative

estimate of the hydrograph translation and attenuation throughout the system. Detailed hydraulic analysis in the vicinity of the site was performed using the reach-scale 2-D model. Therefore, two levee scenarios were evaluated for the unsteady analysis to "bookend" the boundary condition: complete levee failure (*full floodplain*) and the levees remain in place (*levee constrained*).

Sensitivity of both upstream and downstream boundary locations of the 2-D model was tested by changing the Manning's roughness coefficient within a range of 20%. Analysis showed that PMF timing and magnitude at RM 556.16 (2-D model upstream boundary location) were not significantly sensitive to changes in Manning's roughness coefficient. The results demonstrated that the downstream boundary (at RM 510.03) of the 2-D model was sensitive to changes in Manning's roughness coefficient. This highlighted the importance of using Manning's roughness coefficients that had been validated.

In an unsteady simulation, non-conveyed volume would be temporarily stored in the overbank until the bank elevation is exceeded. Therefore, levee features were assigned to prevent water from being stored in the overbank until the channel bank elevation was exceeded. Aerial photography (Reference 2.2-16) was used to determine locations where flow was cut off by natural or man-made blockages.

A summary of the HEC-RAS unsteady computational model parameters are presented in Table 2.2-3.

2.2.3.2.2 *HEC-RAS Unsteady Model Validation*

Several USGS stream gages are located within the modeled reach (Figure 2.2-1). Stage and discharge measurements recorded by USGS at these gage sites during the 2011 flood event were used to generate the inflow boundary conditions of 14 tributaries between RM 810 and RM 498. USACE-reported daily discharges (Reference 2.2-17) at Gavins Point Dam were used to generate the inflow hydrograph at Gavins Point Dam for the validation event and to identify a discharge to be considered as a coincident flow with the PMF. The location of all inflow hydrographs pertaining to this evaluation are presented in Table 2.2-4.

The NSE was evaluated for the predicted stage and discharge hydrographs within the modeled reach (Reference 2.2-18). The HEC-RAS unsteady model was considered validated provided that:

- NSE for each flow hydrograph was greater than 0.90 of the value reported at the gage locations.
- Computed 2011 peak water surface elevations were within ±1 foot of observed WSELs at gage locations.
- Any locations and/or instances that do not meet the above two criteria were justified based on levee breach location and chronology (Reference 2.2-19).

The model was executed using the same roughness coefficients used for the steady-state hydraulic model (Section 2.2.3.2). The flow hydrograph matched reasonably well; however, the stage hydrographs were systemically 1 to 2 ft low. Therefore, in an effort to produce a model that achieves better validation criteria, a simulation with a 10% global increase of Manning's roughness coefficient for

both the channel and overbank was executed. Table 2.2-5 shows the minimum and maximum roughness coefficients.

Graphical comparisons of observed and computed discharge hydrographs and stage measurements at the gage locations were evaluated. The timing and magnitude of predicted flow hydrographs for the 2011 event compared well with the recorded flow hydrograph. The summary of the NSE coefficient along with peak discharge and stage comparisons are shown in Table 2.2-6.

2.2.3.2.3 *PMF Hydraulic Simulations*

The resultant tributary discharges from the final model using PMP Position 3, preceded by the 40% PMP antecedent condition simulation (as discussed in Section 2.2.2) were used in the HEC-RAS unsteady model as lateral inflow discharges at tributary locations throughout the model. For smaller tributaries, contributing flows were aggregated and incorporated at a RM corresponding to the approximate inflow location, and were referred to as Missouri Basin Inflow. Table 2.2-7 summarizes the location of inflow hydrographs used in the model. Additionally, the downstream boundary condition of a normal-depth boundary with a slope of 0.0001 was specified in the model based on sensitivity to validation results.

Two different releases from Gavins Point Dam were considered: the mean monthly average flow of 35,000 cfs and the release of the record flow (from the 2011 flood event, Reference 2.2-17) of 160,000 cfs. Additionally, as discussed in Section 2.2.3.2.1, for each flow condition, two simplifying assumptions were made to provide conservative approximations of PMF translation and attenuation through leveed reaches: full floodplain and levee constrained.

The HEC-RAS unsteady model cross section at RM 556.16 was chosen as the upstream boundary condition of the 2-D model because it is located at a sufficient distance upstream of the point of interest (i.e., CNS) and a tie-back levee, as well as a major confluence (the Nishnabotna River).The downstream boundary condition of the reach-scale 2-D model was located at RM 510.03. This location is approximately 5 miles downstream of federal levee alignments and is in a relatively straight river reach 20 miles downstream of CNS. Analysis of results showed that the full-floodplain condition with 160,000 cfs at Gavins Point Dam results in the highest peak discharge and includes the most conservative Gavins Point Dam release. Therefore, this condition was used for the upstream inflow hydrograph for the 2-D model. The hydrographs, as they translate and attenuate throughout the modeled reach, are shown in Figure 2.2-8. The hydrographs at RM 556.16 are shown in Figure 2.2-9, and the peak magnitude and date are summarized in Table 2.2-8. The rating curves created from the HEC-RAS unsteady model results from both levee conditions were nearly identical, as shown in Figure 2.2-10.

2.2.3.3 Two-Dimensional Reach-Scale Hydraulic Model

The purpose of conducting a detailed 2-D hydraulic model of the Missouri River near CNS was to determine spatially and temporally varying information about water surface elevations, depths, and velocities at CNS during the PMF. Evaluation of site-specific erosion, sedimentation, and debris effects was also performed based on information available from the 2-D model. The modeled (i.e., 2-D) reach

of the Missouri River extended from north of Hamburg, Iowa (approximate RM 556), to north of Craig, Missouri (approximate RM 510). An overview of the modeled reach is shown in Figure 2.2-11. The process used to develop the 2-D model included:

- Developing the digital elevation model (DEM) surface roughness characteristics, appropriate computational mesh and description of levee crests, and boundary conditions from the 1-D unsteady model, as discussed in Section 2.2.3.2.1.
- Validating the 2-D unsteady model with gage data and high-water marks (HWMs) from the 2011 flood event and making any changes necessary to replicate observed stages, as discussed in Section 2.2.3.2.2.
- Analyzing PMF flow scenarios using the 2-D numerical model TUFLOW FV, as discussed in Section 2.2.3.2.3.

2.2.3.3.1 *2-D Model Development*

TUFLOW FV, version 2013.02.056_dev (Reference 2.2-20), was used in this analysis to perform the 2-D hydraulic simulation. The model solves the Non-linear Shallow Water Equations (NLSWE) on a flexible mesh using a finite-volume numerical scheme. The model was not extended away from the Missouri River valley. Specifically, it did not include the Nishnabotna River valley in the model geometry. This eliminates potential storage for floodwaters during the PMF, but the effect on peak WSELs at CNS is insignificant because the capacity of the Missouri River valley is large in comparison with the area extending north along the Nishnabotna River. Near the confluence of the Little Nemaha River and the Missouri River, the study area was extended west of the Missouri River valley approximately four miles. By extending the computational domain, the model was able to better calculate the flow distribution across the levees near the Little Nemaha, which is important for inundation at CNS. Major model inputs included digital elevation data, land cover/land use data, a computational mesh, levee crests, and boundary conditions, as described in the following paragraphs:

- **Elevation Data**: The elevation source for the model was based on the combination of the DEM used for the 1D HEC-RAS steady-state model (Section 2.2.3.1), USGS LiDAR data, USACE bathymetry data, USGS 10-m DEMs (Reference 2.2-21), and a site-specific survey.
- **Surface Roughness**: A combination of land cover/land use data (Figure 2.2-12) from three states, Iowa (Reference 2.2-22), Missouri (Reference 2.2-21), and Nebraska (Reference 2.2-23), were used to assign the appropriate Manning's roughness coefficient representing the surface roughness in the 2-D model.
- **Computational Mesh**: The study area was represented by a computational mesh (Figure 2.2-13 and Figure 2.2-14) containing 410,314 unstructured elements. The mesh was coarser in areas that were farther away from CNS and finer in areas where more detail is required. Buildings at the CNS site were represented by a void space in the computational mesh. The mesh sensitivity analysis demonstrated that changes in mesh size do not cause a large enough change in WSELs to warrant changes from the original mesh size chosen. Most variations were within 0.1 foot at CNS.

- **Levee Crests**: Levee crests and roadway embankment elevations from the DEM were incorporated into the model. It was assumed that no levees, railroad embankments, or roadway embankments would be compromised during the PMF. The autoweir function of TUFLOW FV was used, which automatically applies the standard weir equation to embankments and other high points in the model topography. The weir coefficient was estimated to be on the order of 2.5 using Hager's equation (Reference 2.2-24). A sensitivity analysis was performed and as a result, a weir coefficient of 1.6 was used globally. This represents a conservative WSEL at CNS.
- **Boundary Conditions**: The location and type of the boundary conditions are shown in Figure 2.2-11. The 1-D unsteady HEC-RAS model (as described in Section 2.2.3.2.3) was used to establish the Missouri River boundary condition. The PMF hydrograph (Figure 2.2-9) was divided between within levee (referred to as main channel) and outside of levee discharges east of the main channel (referred to as left overbank). Lateral inflows to the HEC-RAS model at RM 542.10 (Nishnabotna River), RM 535.30 (Honey Creek), and RM 527.8 (Little Nemaha River) were also included. The lateral inflows at RM 535.30 were included with the Missouri River overbank discharge because they represent a basin east of the Missouri River. The inflow hydrographs for each of the four boundaries are shown in Figure 2.2-15.

A normal-depth condition with a friction slope of 0.00022 was used at the downstream end of the model domain based on the geometry in the HEC-RAS unsteady model, which allowed for different main channel and overbank water surface elevations.

Sensitivity analyses were performed for both upstream and downstream boundary conditions. Results showed that WSELs at CNS were not sensitive to the upstream and downstream boundary condition.

2.2.3.3.2 *2-D Model Validation*

The model was validated for the 2011 Missouri River flood event using the available discharge and water level data recorded at USGS gage sites as well as HWMs reported by USACE after action report (Reference 2.2-19). Since several levee breaches were reported during the 2011 flood event, two different steady-state discharge and levee condition combinations were considered in the validation. The first geometry considered levee breaches fully developed for L575 (RM 551 and RM 544) and L550 (RM 539 and RM 523), while the second geometry included only levee breaches fully developed for L575 and not L550. The simulation was conducted using steady-state discharge of 257,000 cfs and 207,000 cfs on the Missouri River for the first and second geometric conditions, respectively. The first case provided the highest discharge on the protected or land side of the levee, while the second case generated the highest discharge on the river side of the levee between RM 544 and RM 523.

Simulated WSELs in the channel and interior were on average 0.1 foot and 0.8 foot lower, respectively, than measured HWMs. Within the context of this validation, the model calculated 36,000 cfs less discharge in the left overbank than was measured on one day during the 2011 flood event, and discharge in the channel is overestimated. Overall, the model simulated flow distribution across the river valley reasonably well for a reach scale model.

2.2.3.3.3 *2-D Model Hydraulic Simulations*

The inflow hydrograph used for the Missouri River was the full floodplain routing of the PMF event with a coincident discharge of 160,000 cfs from Gavins Point Dam, as discussed in detail in Sections 2.2.3.2.3 and 2.2.3.3.1. Figure 2.2-16 shows hydrographs at four locations within the modeled reach. From the upstream end of the model to Hamburg, peak discharges increased due to lateral inflows from the Nishnabotna River. Between the Nishnabotna River and CNS, the peak discharge was attenuated due to storage in the left overbank. Downstream of CNS, peak discharges increased again due to lateral inflows from the Little Nemaha River. The peak PMF discharge at CNS was 797,600 cfs.

Most of the CNS main plant areas were not inundated during the PMF event. Figure 2.2-17 and Figure 2.2-18 present contour plots of the maximum WSEL. Model results showed that the peak WSEL in the main channel near the intake structure was 902.8 ft NAVD88. The peak WSEL at CNS, which occurs at the upstream side of the plant on the main channel side of the levee (see Point B on Figure 2.2-18), was 903.3 ft NAVD88. When WSELs in the right overbank peak, the model indicated shallow flooding with depths of 0.5 foot or less near the Security Building (see Point 3 in Figure 2.2-18).

Maximum water depths are shown in Figure 2.2-19 and Figure 2.2-20 and contours of maximum velocity magnitude are shown in Figure 2.2-21. Peak velocities were less than 0.5 fps across much of the site, with the highest velocities seen where the federal levees north of the site are overtopped. Model results indicated that velocities in that area peak between 5 and 6 fps.

Water depths and velocities are further investigated at three critical locations, including the Intake Structure, Switchyard, and the Elevated Release Point (ERP). A summary of grade elevation, maximum WSEL, maximum velocity, and maximum depth are provided in Table 2.2-9.

2.2.4 Combined Effects

Determination of the total water level at the CNS site with the consideration of wind and wave effects based on the predicted maximum still water level from the PMF (Section 2.2.3.3.3) at the Missouri River is described in the following sections.

2.2.4.1 Water Level at CNS with Wind Setup

In accordance with the guidelines in ANSI/ANS-2.8-1992 (Reference 2.2-8), the maximum PMF level at the plant site needs to consider the wind setup and wave runup effect from the coincidental occurrence of a 2-year design wind event. The 2-year fastest annual mile wind speed at the site is 55 miles per hour (mph) at 30 ft above the ground based on Reference 2.2-8. The methodology outlined in Coastal Engineering Manual (Reference 2.2-25) was followed to convert the annual extreme-mile wind speed to a 1-hour duration wind speed and then to 10-, 15-, and 20-minute wind speed durations.

Wind-driven waves were calculated using the "Windspeed Adjustment and Wave Growth" module of the Automated Coastal Engineering System (ACES) in the Coastal Engineering Design & Analysis System (CEDAS) Version 4.03 (Reference 2.2-26). The ACES uses the wind fetch option, elevation of observed wind, observed wind speed, duration of observed wind, duration of final wind, latitude of

observation, restricted fetch geometry, and average fetch depth as input. The shallow restricted option was chosen because the waves are not expected to propagate under a deep-water condition for a significant duration and the fetch is not unlimited for wave formation. The fetch geometry was determined by extending lines at directional (radial angle) increments of 22.5 degrees from a representative point at the CNS site to the extent of the river cross sections in all directions as shown in Figure 2.2-22. The ACES analysis was completed for each fetch. ACES outputs were spectral significant wave height (H_{m0}) and peak wave period (T_p). Wave characteristics were calculated using equations provided in Reference 2.2-27. Wind approaching along the longest and deepest fetch for each point of interest (Figure 2.2-23) was modeled in ACES to determine the controlling (maximum) significant wave height at each point. In the case that the longest fetch was not the deepest, both the longest fetch and the deepest fetch were checked. The fetch with the larger wave height was then considered to be the controlling fetch.

Wind setup was calculated along the controlling fetch for each point of interest using Equation 4 of U.S. Bureau of Reclamation ACER Technical Memorandum No. 2 (Reference 2.2-28). The calculated value was added to the PMF WSEL (described in Section 2.2.3.3.3) and the new WSEL was used for subsequent wave runup and associated effects calculations. Wind setup was not added to the PMF WSEL on the interior of the perimeter embankments because setup is not significant at these locations.

Equation 2-2 of EM 1110-2-1614 (Reference 2.2-29) was used to convert the spectral significant wave height (H_{m0}) to 1% wave height ($H_{1\%}$, the average of the highest 1% of waves). In accordance with ANSI/ANS-2.8-1992 (Reference 2.2-8), $H_{1\%}$ was used as a *design wave height* for subsequent analyses.

A summary of ACES input and outputs along with wind setup and 1% wave height at six points of interest are presented in Table 2.2-10. The largest wave height was calculated at point of interest West Embankment North (WEmbN, see Figure 2.2-23), with an $H_{1\%}$ of 4.56 ft. Wave heights tended to be largest on the southern and western edges of the site due to the relatively longer and deeper fetches for waves to propagate to those points, whereas waves from the east are blocked by the federal levee.

2.2.4.2 Wave Runup

Locations of runup calculations for waves approaching the [redacted] (b)(3): 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(E) from different directions are shown in Figure 2.2-24. Combinations of Coastal Engineering Manual (CEM) Chapter 6 (Reference 2.2-25) and FEMA guidelines (Reference 2.2-30) were used for wave runup calculations. For conservatism, all reduction factors as well as the factor for influence of angle of incidence were set to 1, meaning no reduction. Design wave height ($H_{1\%}$) was used as wave input. For embankments located north and south of the [redacted] runup calculations showed that these embankments are overtopped. Therefore, Reference 2.2-30 was used to estimate the horizontal extent of runup. The extent of inundation is shown in Figure 2.2-25.

(b)(3): 16 U.S.C. § 824o-1(d), (b)

For WEmbN and North Embankment East (NEmbE) embankments (see Figure 2.2-23), the transmitted wave height was calculated using Equation VI-5-54 of CEM (Reference 2.2-25). This was done because the wave is passing over other embankments to reach these two points of interest. Inputs included transmission coefficient (0.75), design wave height ($H_{1\%}$), median rock size, freeboard

(negative for submerged embankment), and width of embankment crest. The minimum of transmitted wave height and breaker wave height (equal to 0.78 times design wave height [$H_{1\%}$]) was used for runup calculations of these two points. The results are summarized in Table 2.2-11.

Wave runup results are presented in Table 2.2-12. The wave runup from the west was calculated to be 1.4 ft, contributing to a maximum water level of 902.9 ft NAVD88. The runup produced from the plant west is not expected to reach the main critical building complex, as shown in the Figure 2.2-25. Runup from the plant north was calculated to be 0.5 foot, resulting in a maximum water level of 904.1 ft NAVD88. Topography indicates that CNS grounds in this area are below 904.1 ft NAVD88, further indicating that the buildings in this area would be exposed to runup from interior waves. Any interior protection provided to prevent exposure to wave runup from the north should extend to an elevation of at least 904.1 ft NAVD88 to provide protection to the Main Building Complex. While the farthest extent of the runup is expected to reach the Main Building Complex, it is expected to be minimal (on the order of inches). Reference 2.2-31 indicates that plant flood barriers extend to an elevation of 906 ft Plant Datum (906.37 ft NAVD88), which is high enough to provide adequate protection from the runup.

The vertical extent of runup on the Intake Structure was calculated to be 908.4 ft NAVD88, which is equivalent to a PMF WSEL of 903.0 ft NAVD88 plus 5.4 ft runup.

2.2.5 Associated Flooding Impacts

2.2.5.1 Overtopping

Equation II-4-28 of CEM (Reference 2.2-25) was used to estimate a theoretical wave runup at an unsubmerged embankment, identified as point of interest West Embankment South (WEmbS) in Figure 2.2-23. Inputs included design wave height ($H_{1\%}$) at the embankment toe, peak wave period, embankment slope, embankment freeboard, and surface roughness reduction factor. Analysis showed that the theoretical runup is higher than the embankment height, confirming that the embankment is overtopped. Equation VI-5-22 of CEM (Reference 2.2-25) was used to calculate the overtopping discharge at point of interest WEmbS. An overtopping discharge of 45 gallons per minute per linear foot was calculated at this location.

2.2.5.2 Erosion and Sedimentation

The potential for erosion of the embankments due to wave action was assessed using erosion threshold guidance from the USACE ERDC/CHL TR-10-7 (Reference 2.2-32). Inputs included a safety coefficient, grass quality factor, design wave height ($H_{1\%}$), embankment slope, and duration of waves. The potential maximum erosion depth was calculated to be 0.26 foot. This erosion depth was based on the wave exposure at WEmbN, which is the largest (most conservative) wave height as shown in Table 2.2-10.

2.2.5.3 Hydrodynamic Forces

Hydrostatic and wave-induced hydrodynamic pressures and forces were calculated on the Intake Structure in accordance with the Goda pressure formula as described in CEM Table VI-5-53

(Reference 2.2-25). Inputs included design wave height, wave length, water depth, and structure geometry. The largest wave heights are expected to impact the east face of the Intake Structure. For conservatism, pressure and force calculations were computed on the east face and treated as applicable on the north and south faces.

The Morison equation (Reference 2.2-27) was used to calculate the forces on the north wall of the Intake Structure due to a river current. Inputs included a drag coefficient, still water elevation, bottom elevation, water density, and current velocity. Outputs included the force per unit width of the north face wall of the Intake Structure.

Values for hydrostatic, wave-induced hydrodynamic, and current-induced hydrodynamic forces are listed in Table 2.2-13.

2.2.5.4 Debris and Impact Loads

Review of possible flood debris items was undertaken to develop a spectra of flood debris sources. The following items were considered to represent a range of debris sources available upstream of CNS. Two debris sources and masses were based on ASCE 7-10 (Reference 2.2-33) recommendations and the barge mass was based on typical barge sizes on the Missouri River. Additional vehicle and marine vessel masses were based on manufacturer data available on the Internet.

- ASCE 7-10 miscellaneous debris - 1 kip
- Large natural debris (ice, trees) - 4 kips
- Large vehicles and boats (tug boat, bus, mobile home) - 40 kips
- Tank-type debris (train cars, chemical tanks, semi trailers) -100 kips
- Barge - 5,100 kips

Flood debris impact loads were calculated using the methodology presented in Chapter C5 of ASCE 7-10 (Reference 2.2-33), with the following considerations:

- Duration of impact load of 0.03 seconds was recommended based on Reference 2.2-33, Section C5.4.4, with the pulse shape taken as a half sine wave.
- Velocity of the debris was considered to be equal to the water velocity, which may differ for each debris type depending on the expected debris path.
- Importance factor value of 1.3 for Risk Category IV was used (Reference 2.2-34, Section 4.2.8).
- Depth coefficient was selected from Table C5-2 of Reference 2.2-33.
- Orientation coefficient value of 0.80 was used, as recommended in Equation C-5 of Reference 2.2-33.
- Blockage coefficient from Table C5-3 of Reference 2.2-33, considering sheltering within 100 ft upstream was used.

- Dynamic load factor depends on the fundamental vibration period of the impacted structure, and the maximum value from Table C5-4 of Reference 2.2-33 was conservatively used.

Estimated debris impact loads due to PMF for different debris were determined using the input values identified above and based on channel and overbank velocities. A summary of impact load values is presented in Table 2.2-14. As shown in the table, a barge with the maximum channel velocity (8.5 fps) will result in a maximum impact load of 79,239 kips.

2.2.6 References

2.2-1. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

2.2-2. United States Geological Survey (USGS), 2013, USGS National Water Information System (NWIS), Washington, D.C., accessed June 1, 2013, http://waterdata.usgs.gov/nwis/.

2.2-3. Missouri River Annex, 2011, Mississippi River and Tributaries Waterways Action Plan.

2.2-4. U.S. Army Corps of Engineers (USACE). August 2010. Hydrologic Modeling System HEC-HMS Version 3.5. U.S. Army Corps of Engineers, Hydrologic Engineering Center: Davis, California.

2.2-5. U.S. Army Corps of Engineers (USACE). January 2010. River Analysis System HEC-RAS Version 4.1. USACE, Hydrologic Engineering Center, Davis, California.

2.2-6. United States Department of Agriculture, 2012. Geospatial Data Gateway. Website: http://datagateway.nrcs.usda.gov/, accessed 12/14/2012. Washington, D.C.

2.2-7. ArcMap Version 10.2 (2013). ESRI, Redlands, CA.

2.2-8. American Nuclear Society, 1992, ANSI/ANS-2.8-1992: Determining Design Basis Flooding at Power Reactor Sites, American Nuclear Society Publishing, La Grange Park, IL.

2.2-9. National Oceanic and Atmospheric Administration (NOAA). Atlas 14 Volume 8 Version 2, Precipitation-Frequency Atlas of the United States, Midwestern States, NOAA, National Weather Service, Silver Spring, MD.

2.2-10. NOAA, 1978. Hydrometeorological Report No. 51, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, U.S. Department of Commerce, NOAA, USACE, Washington, D.C.

2.2-11. NOAA, 1982, Hydrometeorological Report No. 52, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, U.S. Department of Commerce, NOAA, USACE, Washington, D.C.

2.2-12. Nebraska Department of Natural Resources (NDNR), 2013, Department of Natural Resources Stream Gaging, accessed June 1, 2013, http://dnr.ne.gov/docs/hydrologic2013.html.

2.2-13. U.S. Army Corps of Engineers (USACE). March 2000. Hydrologic Modeling System HEC-HMS Technical Reference Manual. U.S. Army Corps of Engineers, Hydrologic Engineering Center: Davis, California.

2.2-14. U.S. Army Corps of Engineers (USACE), November 2003, Upper Mississippi River System Flow Frequency Study; Hydraulics and Hydrology Appendix F, Missouri River, USACE, Omaha District, Omaha, NE.

2.2-15. Federal Emergency Management Agency (FEMA). May 3, 2010. *Flood Insurance Study; Douglas County, Nebraska and Incorporated Areas*. Flood Insurance Study Number 31055CV000C. U.S. Department of Homeland Security, Washington, DC.

2.2-16. United States Geological Survey (USGS), 2013, USGS LandsatLook Viewer, Washington, D.C., accessed October 1, 2013. http://landsatlook.usgs.gov/.

2.2-17. U.S. Army Corps of Engineers (USACE), 2013, USACE Northwestern Division MRR Daily River Bulletin, accessed September 1, 2013. http://www.nwd-mr.usace.army.mil/rcc/.

2.2-18. Moriasi, D.N., J.G. Arnold, M.W. Van Liew, R.L. Bingner, R.D. Harmel, and T.L. Veith, 2007, "Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watershed Simulations," Transactions of the ASABE 50(3): 885–900, http://ddr.nal.usda.gov/dspace/bitstream/10113/9298/1/IND44003774.pdf.

2.2-19. U.S. Army Corps of Engineers (USACE), undated, After Action Report: Missouri River and Tributaries Flood of 2011, Received from John Remus, USACE, via e-mail on October 1, 2013.

2.2-20. BMT WBM, 2013, TUFLOW FV User Manual. Flexible Mesh Modeling, Version 2013.02.056_dev.

2.2-21. University of Missouri, Columbia, 2013, Missouri Spatial Data Information Service, accessed October 10, 2013. ftp://www.cares.missouri.edu/pub/dem/.

2.2-22. Iowa Department of Natural Resources, 2013, Natural Resources Geographic Information Systems Library, accessed November 4, 2013. http://www.igsb.uiowa.edu/nrgislibx/.

2.2-23. Multi-Resolution Land Characteristics Consortium, 2013, National Land Cover Database, accessed May 17, 2013. http://www.mrlc.gov/index.php.

2.2-24. Hager, W.H., 1987. "Lateral Outflow Over Side Weirs." Journal of Hydraulic Engineering, ASCE, Vol. 113, No. 4, PP 491-504.

2.2-25. U.S. Army Corps of Engineers (USACE), 2008, Coastal Engineering Manual, EM 1110-2-1100, Washington, D.C. (in 6 volumes).

2.2-26. CEDAS-ACES Version 4.03 (2014), Veri-Tech, Vicksburg, MS.

2.2-27. Dean, R. and R. Dalrymple, 1991, Water Wave Mechanics for Engineers and Scientists. World Scientific, Hackensack, NJ.

2.2-28. United States Bureau of Reclamation, 1981, Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams, ACER Technical Memorandum No. 2.

2.2-29. U.S. Army Corps of Engineers (USACE), 1995, Engineer Manual 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads.

2.2-30. Federal Emergency Management Agency, 2007, Guidelines and Specifications for Flood Hazard Mapping Partners, Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update, 360 p.

2.2-31. Nebraska Public Power District CNS Operations Manual, Maintenance Procedure 7.0.11, Rev. 29, "Flood Control Barriers."

2.2-32. U.S. Army Corps of Engineers (USACE), 2010, ERDC/CHL TR-10-7, Flood-Side Wave Erosion of Earthen Levees: Present State of Knowledge and Assessment of Armoring Necessity.

2.2-33. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures."

2.2-34. United States Nuclear Regulatory Commission (NRC), July 29, 2013, Guidance for Assessment of Flooding Hazards Due to Dam Failure, Japan Lessons-Learned Project Directorate JLD-ISG-2013-01.

2.2-35. U.S. Army Corps of Engineers (USACE), August 1994, EM 1110-2-1417 Flood Runoff Analysis, U.S. Army Corps of Engineers: Washington, D.C.

2.2-36. Natural Resources Conservation Service (NRCS) and United States Department of Agriculture (USDA), 2013, Soil Survey Geographic (SSURGO) Databases for Iowa, Kansas, Minnesota, Missouri, Nebraska, North Dakota, and South Dakota, accessed April 30, 2013. http://soildatamart.nrcs.usda.gov.

2.2-37. U.S. Department of Agriculture (USDA), June 1986, Technical Release 55.Urban Hydrology for Small Watersheds.

2.2-38. Natural Resources Conservation Service (NRCS), May 2010, National Engineering Handbook, Part 630 Hydrology.

2.2-39. Chow, V.T., 1959, Open Channel Hydraulics, McGraw Hill.

2.2.7 Tables

Tables associated with Section 2.2 are presented on the following pages.

Table 2.2-1: Point 3 (shown in Figure 2.2-4) Adjusted Precipitation Depths Based on HMR 52

6-hr Increment Number	12	11	10	9	7	6	5	3	1	2	4	8
6-hr Time Period	72	66	60	54	42	36	30	18	6	12	24	48
Time Period	0-6hr	6-12hr	12-18hr	18-24hr	24-30hr	30-36hr	36-42hr	42-48hr	48-54hr	54-60hr	60-66hr	66-72hr
Isohyet. (in)												
A (10)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.69	12.52	2.18	0.65	0.64
B (25)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.69	11.67	2.13	0.65	0.64
C (50)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.68	10.87	2.08	0.65	0.64
D (100)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.68	9.95	2.05	0.65	0.64
E (175)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.67	9.19	2.01	0.65	0.64
F (300)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.67	8.43	1.98	0.65	0.64
G (450)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.66	7.90	1.94	0.65	0.64
H (700)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.66	7.26	1.91	0.65	0.64
I (1000)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.66	6.74	1.88	0.65	0.64
J (1500)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.66	6.18	1.86	0.65	0.64
K (2150)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.65	5.70	1.82	0.65	0.64
L (3000)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.65	5.26	1.80	0.65	0.64
M (4500)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.65	4.69	1.77	0.65	0.64
N (6500)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.64	4.17	1.73	0.65	0.64
O (10000)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.64	3.57	1.70	0.65	0.64
P (15000)	0.30	0.30	0.30	0.30	0.64	0.64	0.64	0.63	2.97	1.68	0.65	0.64
Q (25000)	0.20	0.20	0.20	0.20	0.42	0.42	0.42	0.43	1.44	0.95	0.43	0.42
R (40000)	0.04	0.04	0.04	0.04	0.08	0.08	0.08	0.08	0.32	0.12	0.08	0.08

Table 2.2-2: Model Input Parameter Definitions and References

Hydrologic Parameter	Definition	Reference
Drainage area	Area that contributes to runoff.	USGS gages and GIS tools
Deficit and Constant Loss Parameters		
Initial deficit	The amount of precipitation lost due to interception and depression storage.	Table 6-1 of EM 1110-2-1417 (Reference 2.2-35), as referenced in HEC-HMS Technical Reference Manual (Reference 2.2-13)
Constant loss rate	The loss rate is based on the maximum infiltration capacity of the soil.	Table 11 of HEC-HMS Technical Reference Manual (Reference 2.2-13)
Maximum deficit	The maximum amount of water that can be stored in the soil profile.	SSURGO GIS soil data available water holding capacity (Reference 2.2-36)
Impervious surface	The percentage of the basin from which all precipitation runs off.	Aerial photographs
ModClark Transform Parameters		
Time of concentration	The time it takes flow to travel from the hydraulically most distant point to the watershed outlet.	TR-55 method (Reference 2.2-37). Slope was calculated using elevation and flowpath length values taken from USGS topographic map; velocity for shallow concentrated flow was estimated using NEH-4 (Reference 2.2-38).
Storage coefficient	An index of the temporary storage of runoff in the watershed as it drains to the outlet point. This parameter typically is based on gage data and is adjusted during the calibration process.	HEC-HMS Technical Reference Manual (Reference 2.2-13). It is assumed that this value was adjusted through calibration by USACE.

Table 2.2-2: Model Input Parameter Definitions and References, continued

Hydrologic Parameter	Definition	Reference
Recession Baseflow Parameters		
Initial discharge	Discharge at the start of the simulation. These values may be adjusted based on antecedent conditions. If gage data warrant, this is adjusted for historic storms. ANSI/ANS-2.8-1992 suggests use of mean monthly baseflow at the beginning of an antecedent storm, during the month of occurrence, for the PMF hydrograph analysis.	USGS and NDNR gages (Reference 2.2-2 and Reference 2.2-12)
Baseflow recession constant	Represents the decay of the baseflow. This parameter may be adjusted during the calibration process.	Table 16 of HEC-HMS Technical Reference Manual (Reference 2.2-13)
Threshold type (ratio to peak or threshold discharge)	User-defined threshold flow that defines the time at which the recession model of HEC-HMS defines the total flow, expressed as either a flow rate or ratio to the computed peak. It was assumed that this value was adjusted through calibration by USACE.	HEC-HMS Technical Reference Manual (Reference 2.2-13)
Reservoir Parameters		
Stage-Discharge Function	Stage-discharge relationship for the reservoir outlet works.	Provided by USACE
Elevation-Storage Function	Elevation-storage relationship of the reservoir.	Provided by USACE
Initial Elevation	Initial elevation in the reservoir at the start of the simulation.	Provided by USACE

Table 2.2-3: Summary of Unsteady Computational Parameters

Parameter	Value
Time step (minutes)	5
Theta (implicit weighting factor)	1.0
Water surface calculation tolerance (ft)	0.02
Maximum number of iterations	20
DSS messaging level	4
Maximum error in water surface solution (abort tolerance, ft)	100

Table 2.2-4: Location of Inflow Hydrographs

Inflow Hydrograph (USGS Gage)	Inflow River Mile [1]
Gavins Point Dam	810.87
James River near Scotland, SD (06478500)	797.73
Vermillion River near Vermillion, SD (06479010)	771.90
Big Sioux River at Akron, IA (06485500)	734.00
Floyd River at James, IA (06600500)	730.95
Omaha Creek at Homer, NE (06601000)	719.62
Little Sioux River near Turin, IA (06607500)	669.23
Soldier River at Pisgah, IA (06608500)	664.00
Boyer River at Logan, IA (06609500)	635.22
Platte River at Louisville, NE (06805500)	594.82
Weeping Water Creek at Union, NE (06806500)	568.67
East Nishnabotna River at Riverton, IA (06809900) and West Nishnabotna River near Riverton, IA (06808820)	542.10
Little Nemaha River at Auburn, NE (06811500)	527.80
Tarkio River at Fairfax, MO (06813000)	507.50

Note:
1. River Mile corresponding to location in HEC-RAS model.

Table 2.2-5: Comparison of Manning's Roughness Coefficients used in the Model to Standard Values

	Channel		Overbank	
Parameter	**Minimum**	**Maximum**	**Minimum**	**Maximum**
Values from steady-state model	0.025	0.029	0.020	0.120
Global 10% roughness increase	0.028	0.032	0.022	0.132
Values from Reference 2.2-39	0.016	0.060	0.011	0.160

Table 2.2-6: NSE Coefficient Summary and Peak Discharge Comparison for 2011 Validation Simulation

Location	Nash-Sutcliffe Efficiency Coefficient	Observed 2011 Peak Discharge (cfs)	Predicted Peak Discharge (cfs)	Observed 2011 Peak Stage (ft, NAVD88)	Predicted Peak Stage (ft, NAVD88)
USGS gage 06486000, Missouri River at Sioux City, IA (RM 732.2)	0.993	192,000	185,484	1092.74	1092.01
USGS gage 06601200, Missouri River at Decatur, NE (RM 691.0)	0.993	191,000	186,390	1050.40	1049.78
USGS gage 06610000, Missouri River at Omaha, NE (RM 615.9)	0.987	217,000	199,408	984.53	984.29
USGS gage 06807000, Missouri River at Nebraska City, NE (RM 562.6)	0.981	229,000	227,608	933.86	933.14
USGS gage 06810070, Missouri River at Brownville, NE (RM 535.3)	0.941	272,000	233,089	904.82	903.51
USGS gage 06813500, Missouri River at Rulo, NE (RM 498.0)	0.943	328,000	234,043	863.67	863.94

Table 2.2-7: Inflow Hydrograph Location Summary for PMF Simulations

Inflow Name	Lateral Flow River Mile [1]
Missouri Basin Tributary [2]	808.26
Missouri Basin Tributary [2]	807.02
James River	797.73
Missouri Basin Tributary [2]	796.30
Bow Creek [2]	787.75
Missouri Basin Tributary [2]	775.27
Vermillion River	771.90
Missouri Basin Tributary [2]	751.61
Aowa Creek [2]	745.19
Elk Creek [2]	737.40
Big Sioux River	734.00
Perry Creek	731.76
Floyd River	730.95
Omaha Creek	719.62
Blackbird Creek [2]	697.41
Missouri Basin Tributary [2]	808.26
Missouri Basin Tributary [2]	807.02
James River	797.73
Missouri Basin Tributary [2]	796.30
Bow Creek [2]	787.75
Missouri Basin Tributary [2]	775.27
Vermillion River	771.90
Missouri Basin Tributary [2]	751.61
Aowa Creek [2]	745.19

Table 2.2-7: Inflow Hydrograph Location Summary for PMF Simulations, continued

Inflow Name	Lateral Flow River Mile [1]
Elk Creek [2]	737.40
Big Sioux River	734.00
Perry Creek	731.76
Floyd River	730.95
Omaha Creek	719.62
Blackbird Creek [2]	697.41
Missouri Basin Tributary [2]	691.35
Little Sioux River	669.23
Tekamah Ditch [2]	664.16
Lower Soldier River	664.00
Missouri Basin Tributary [2]	648.78
Boyer River	635.22
Pigeon Creek [2]	622.16
Missouri Basin Tributary [2]	616.48
Mosquito Creek [2]	605.87
Platte River	594.82
Pony Creek and Keg Creek [2]	587.06
Plum Creek and Ervine Creek [2]	572.05
Missouri Basin Tributary [2]	561.93
South Branch Weeping Water Creek	568.67
Nishnabotna River	542.10
Honey Creek [2]	535.30
Little Nemaha River	527.80
Tarkio River	507.50

Notes:

1. River Mile corresponding to location in HEC-RAS model.
2. These tributaries are not listed in Table 2.2-4 because they are subbasins within the Missouri River basin HMS model that do not have USGS stream gages.

Table 2.2-8: PMF Simulations Peak Magnitude and Time at RM 556.16

Simulation	Peak Discharge (cfs)	Peak Time – Event Begins on 1/1 00:00 (date, hours : minutes)
PMF + 35,000 cfs at Gavins Point, full floodplain	522,400	1/10 22:00
PMF + 35,000 cfs at Gavins Point, levee constrained	546,700	1/10 15:00
PMF + 160,000 cfs at Gavins Point, full floodplain	731,200	1/10 15:00
PMF + 160,000 cfs at Gavins Point, levee constrained	712,700	1/10 11:00

Table 2.2-9: Monitoring Point Summary

Monitoring Point	Grade Elevation (ft NAVD88)	Maximum WSEL (ft NAVD88)	Maximum Overland Velocity (fps)	Maximum Channel Velocity (fps)	Maximum Depth (ft)
1. Intake structure (north)	894.0	902.8	0.4	8.5	8.8
2. Intake structure (east)	880.0	902.8	2.0	8.5	22.8
3. Switchyard	896.6	901.5	0.5	N/A	4.9
4. ERP	892.6	902.8	0.2	N/A	10.2

Table 2.2-10: Calculation of Wind-Driven Waves and Wind Setup Approaching CNS

Point of Interest	Controlling Fetch Angle (deg)	Average Depth Along Fetch (ft)	Fetch Length (ft)	Design Wind Speed (mph)	Wind Speed Duration (min)	H_{m0} (ft)	T_p (s)	Wavelength (ft)	Wind Setup (ft)	$H_{1\%}$ (ft)
Intake structure	0	15.09	10466	46.8	10	2.16	2.68	36.4	0.21	3.61
NEmbE	0	14.33	12665	46.1	15	2.23	2.74	37.8	0.25	3.72
NEmbW	337.5	12.55	6220	46.8	10	1.66	2.32	27.4	0.15	2.77
SEmb	180	16.09	19525	45.6	20	2.72	3.07	47.0	0.34	4.54
WEmbN	180	16.23	19948	45.6	20	2.73	3.08	47.3	0.35	4.56
WEmbS	180	16.50	19311	45.6	20	2.71	3.06	46.8	0.33	4.53

Table 2.2-11: Submerged Embankment Results

Point of Interest	Freeboard, including Setup (ft)	Transmission Coefficient	Transmitted Wave Height (ft)	Maximum Depth-Limited Wave (ft)
WEmbN	-7.4	0.75	3.42	5.77 transmitted wave height acceptable
NEmbE	-2.2	0.08	0.30	1.48 transmitted wave height acceptable

Table 2.2-12: Wave Runup Approaching CNS Main Building Complex

Point of Interest	Wave Runup (ft)	PMF WSEL (ft NAVD88)[1]	Runup Vertical Extent (ft NAVD88)	Runup Horizontal Extent (ft)
WEmbN	1.4	901.5	902.9	N/A
NEmbE	0.5	903.6	904.1	N/A
N of IS	4.9	903.0	N/A	103
S of IS	6.0	902.7	N/A	128

Note:

1. Wind setup is included in the PMF values listed for N of IS and S of IS. Wind setup was not included in the PMF values listed for WEmbN and NEmbE due to the wave calculations being applied to a location that is within the perimeter embankments where wind setup would be negligible.

Table 2.2-13: Forces on Intake Structure

Type	Resultant Force Elevation (ft NAVD88)	Force (lb per linear foot of wall)
Hydrostatic	869.6	78,312
Wave-induced	888.1	3,752
Current-induced	878.0	25

Table 2.2-14: Debris Impact Loads

Maximum Velocity	Debris Source	PMF (kip)
Channel (8.5 fps)	1000# miscellaneous debris	15.54
	4000# large natural debris	62.15
	Large vehicles and boats	621.48
	Tank-type debris	1,553.71
	Barge	79,239
Overland (2.3 fps)	1000# miscellaneous debris	4.20
	4000# large natural debris	16.82
	Large vehicles and boats	168.17
	Tank-type debris	420.41
	Barge	21,441

2.2.8 Figures

Figures associated with Section 2.2 are presented on the following pages.

Figure 2.2-1: Location Map

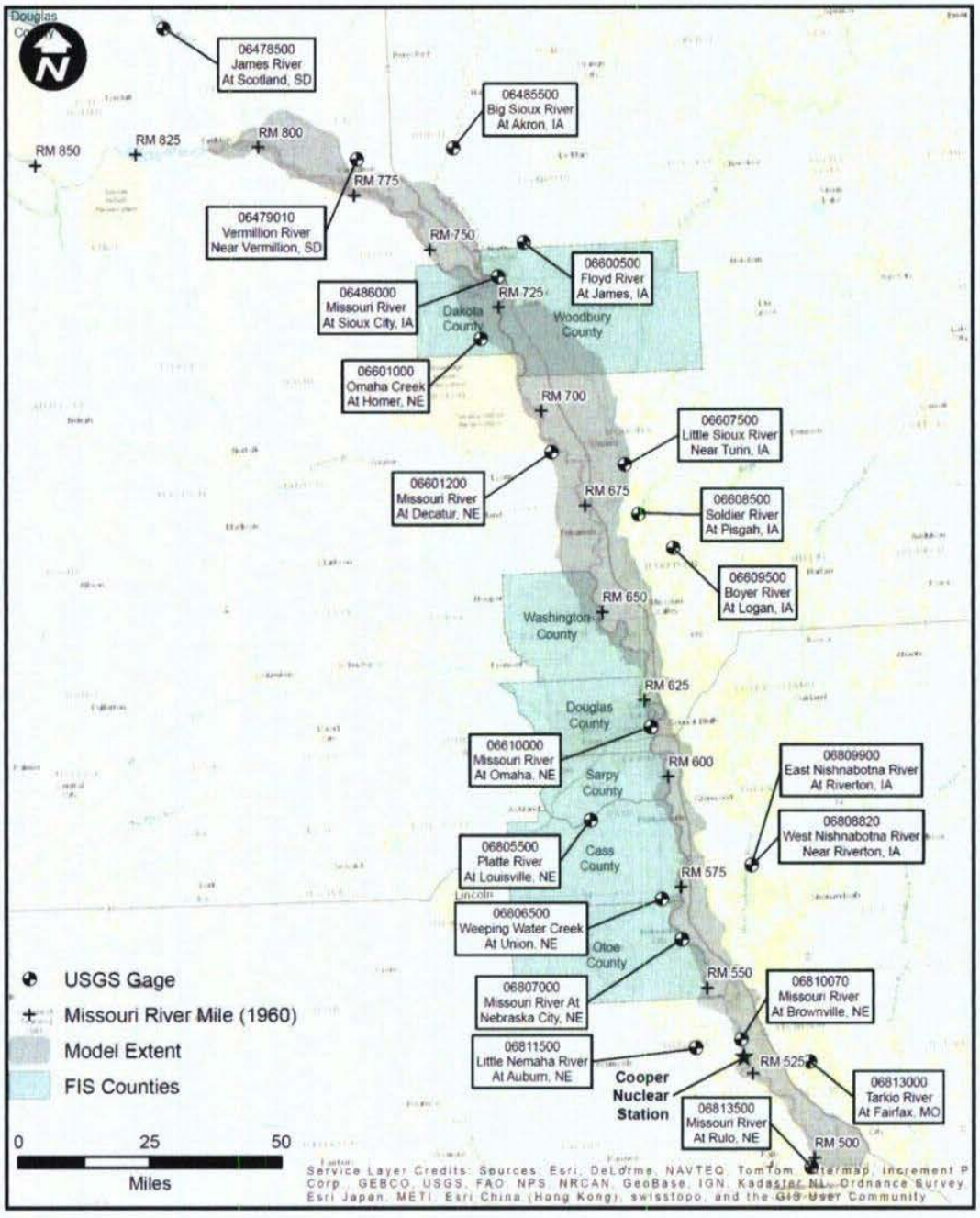


Figure 2.2-2: PMP Position 1 - Sheet a (see subsequent Sheets b, c, and d for PMP Positions 2 to 4)

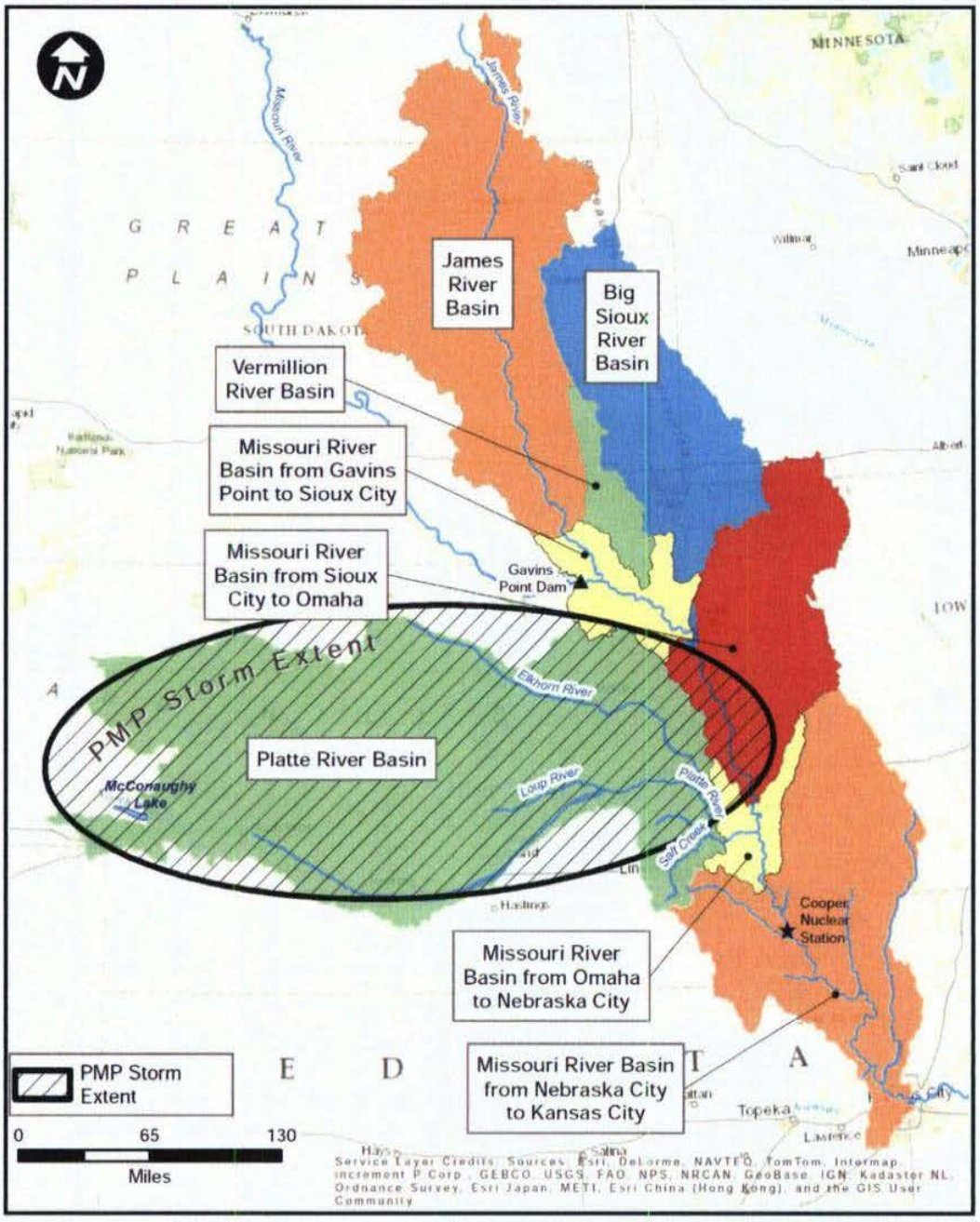


Figure 2.2-2: PMP Position 2 (sheet b)

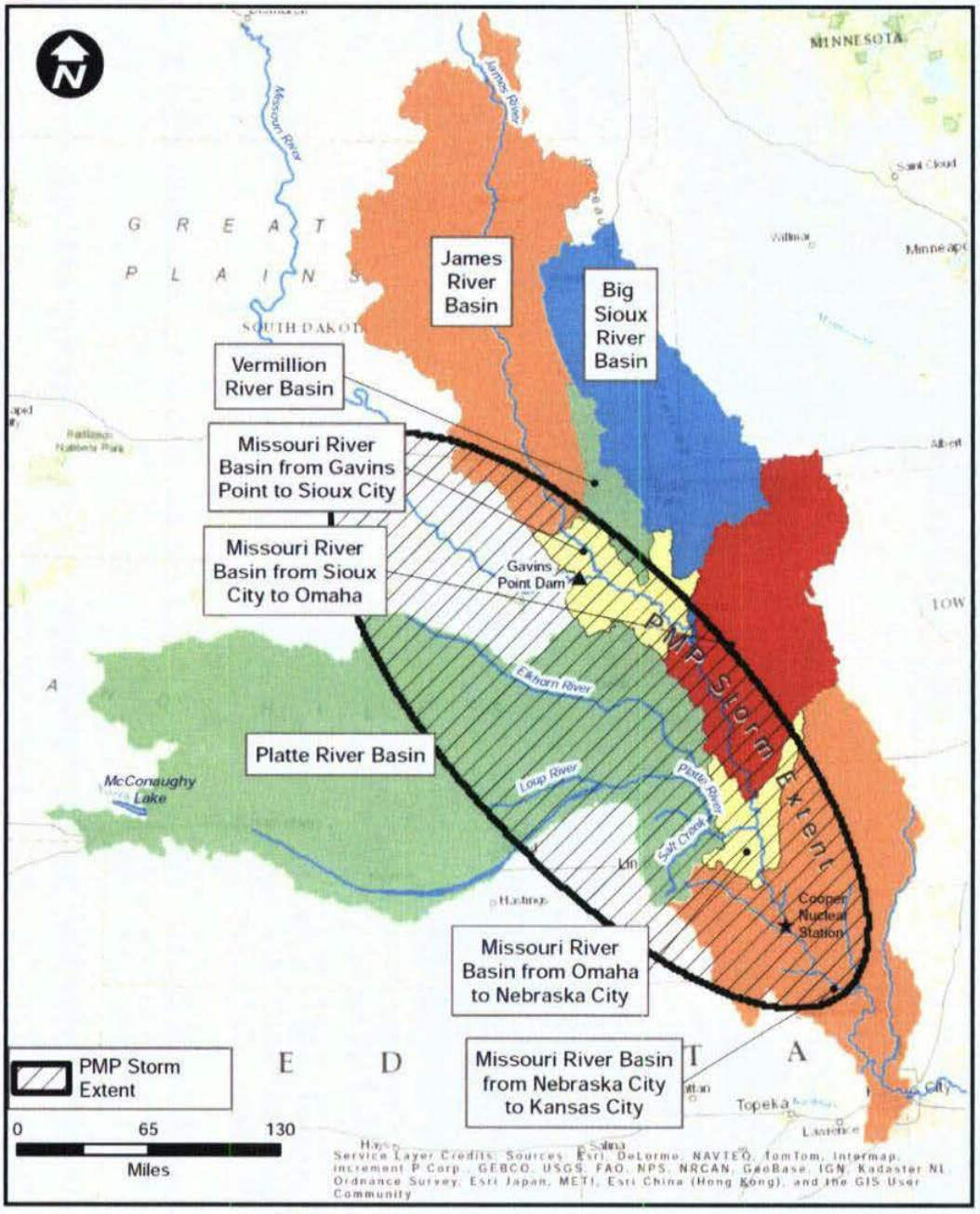


Figure 2.2-2: PMP Position 3 (sheet c)

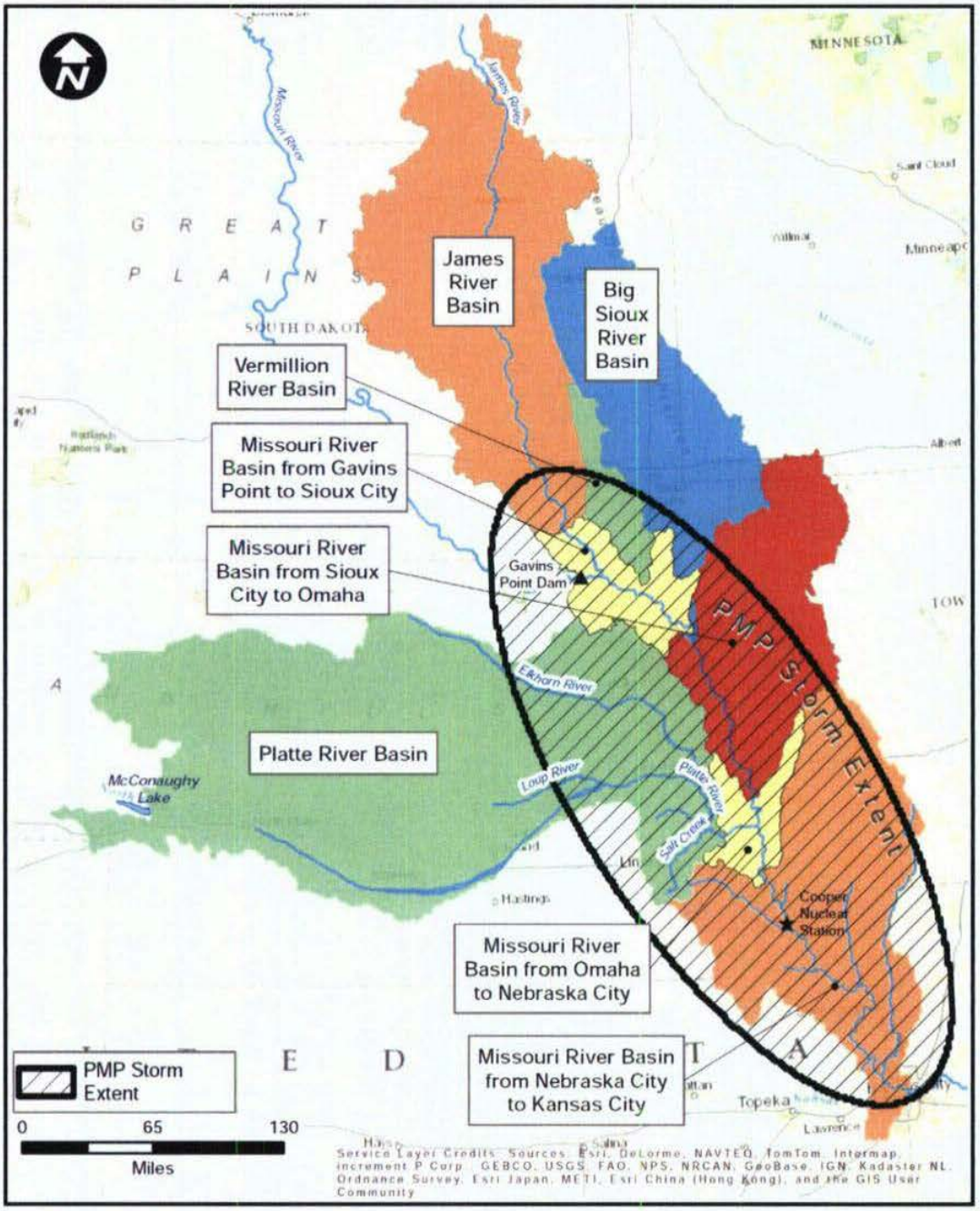


Figure 2.2-2: PMP Position 4 (sheet d)

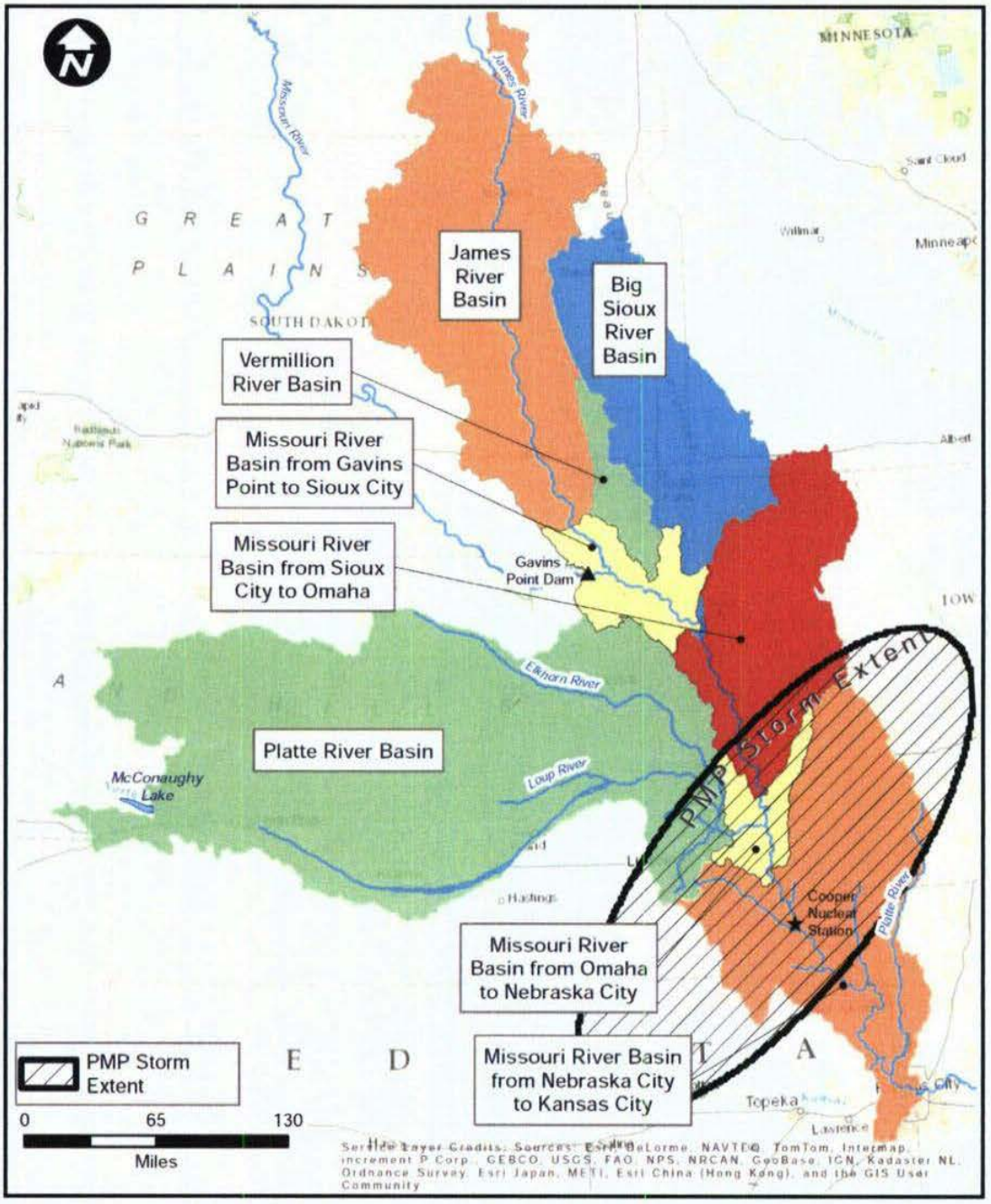


Figure 2.2-3: Geographic Distribution of Basins of Influence for Missouri River Drainage above CNS

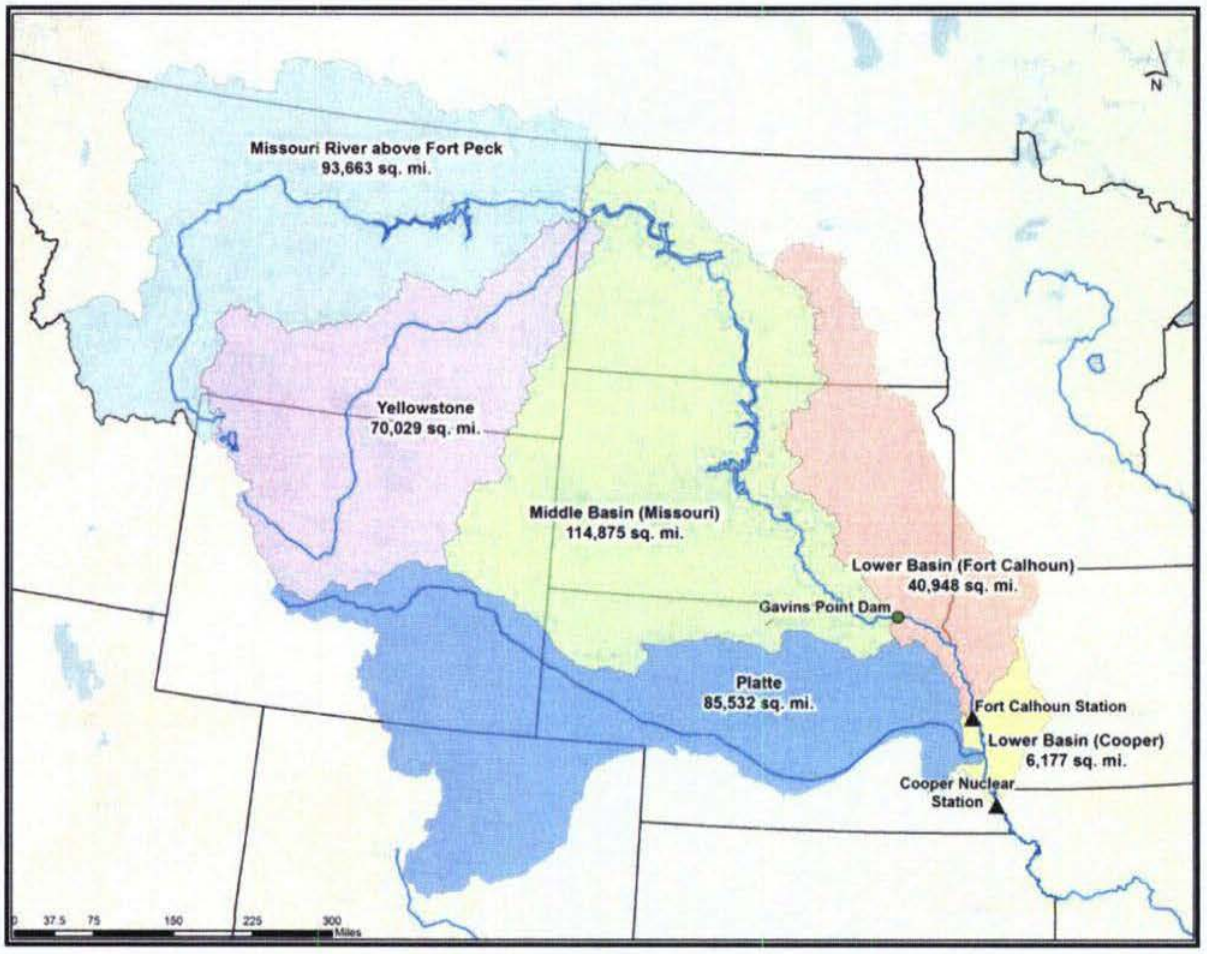


Figure 2.2-4: PMP Spatial Distribution over Platte River Basin (P3 Centroid; 41.35N, 96.20W)

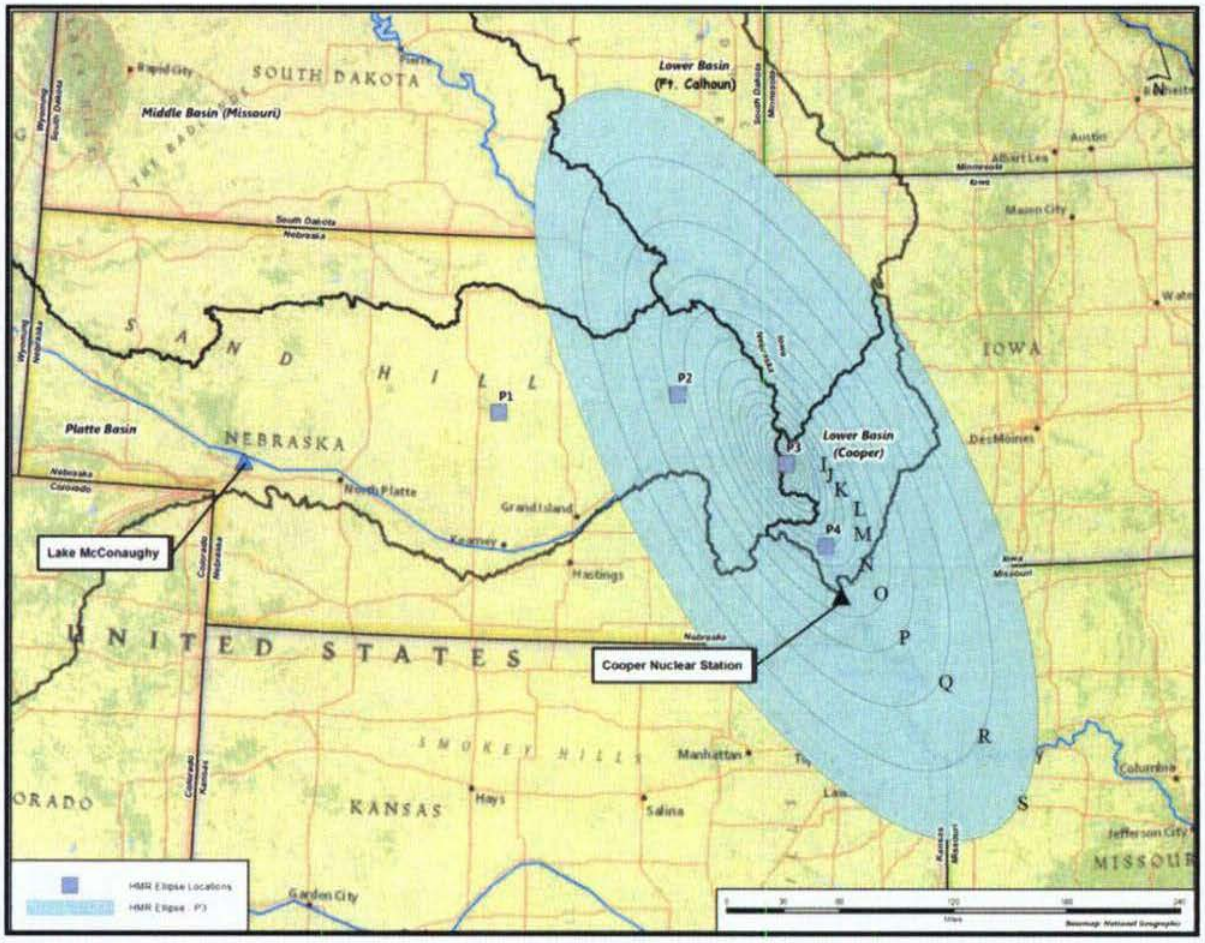
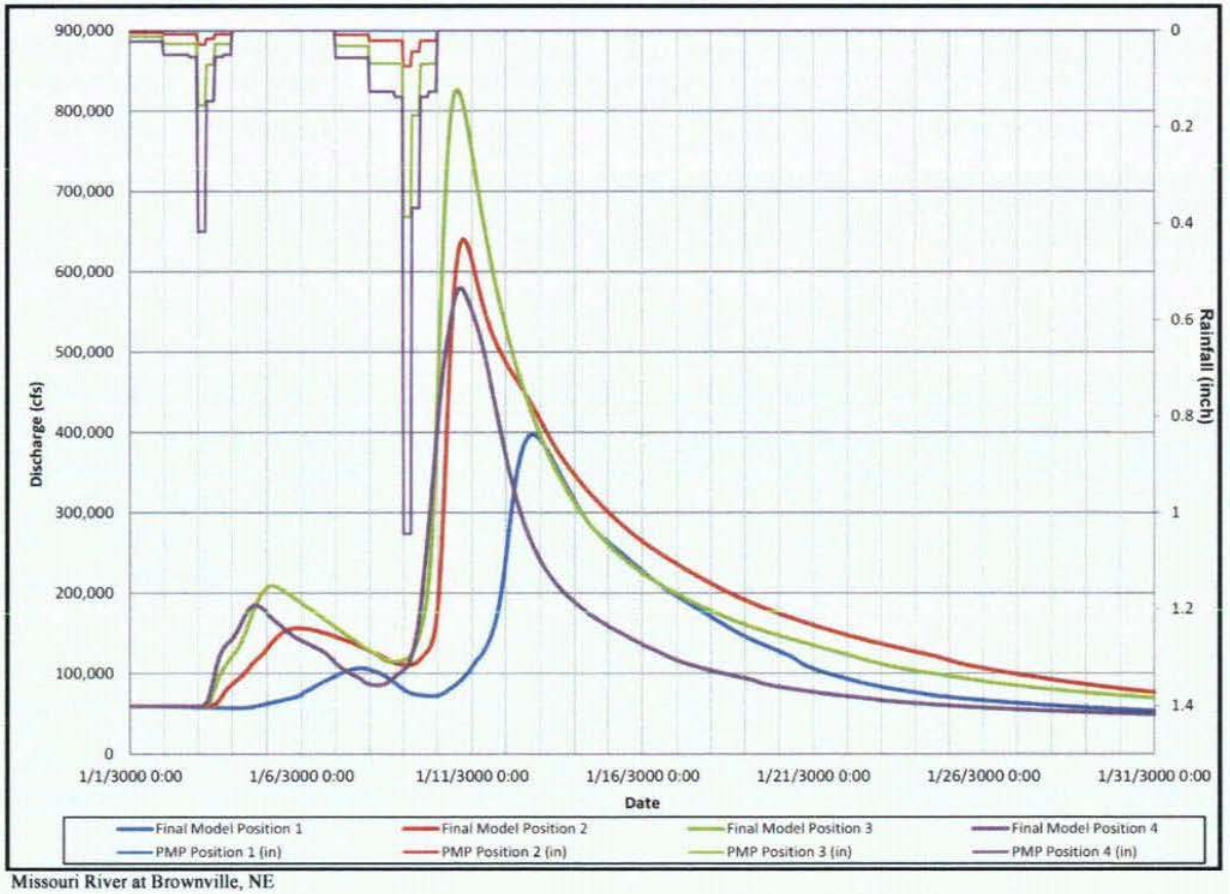
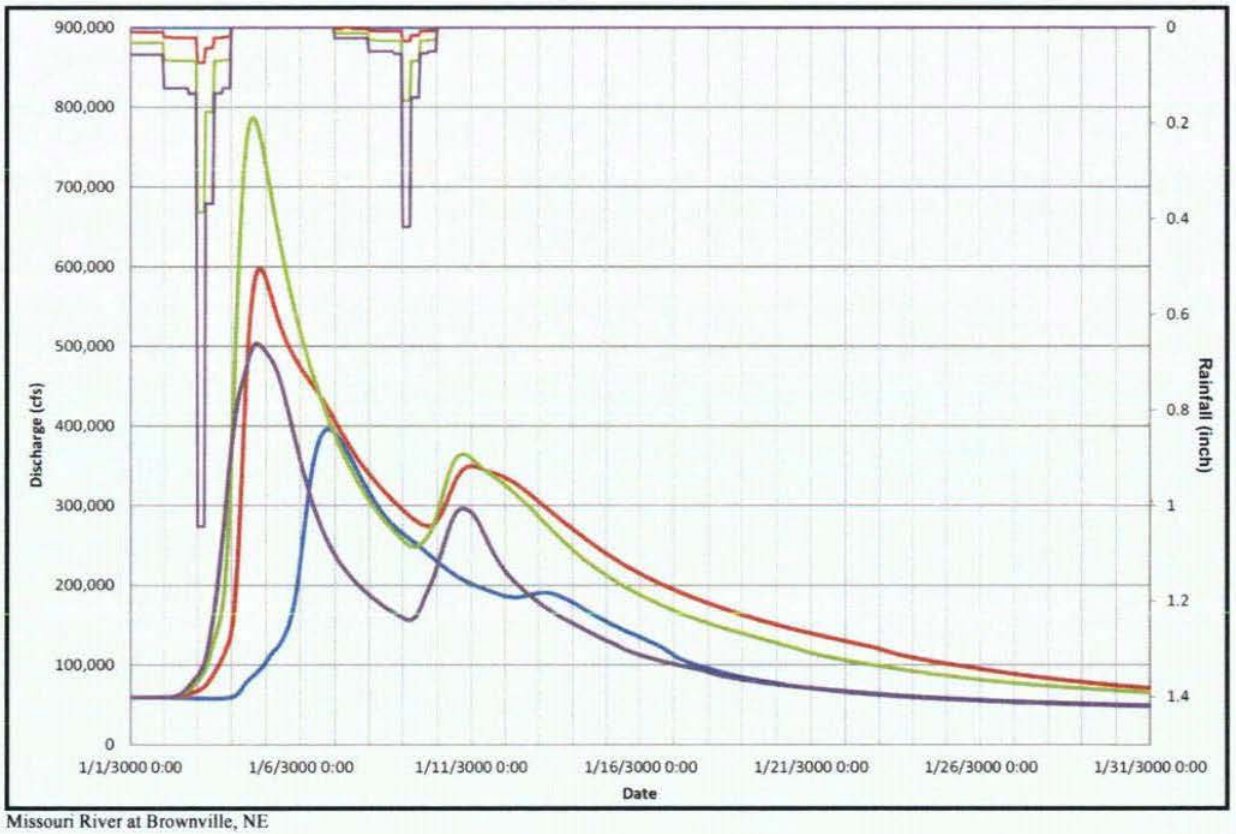


Figure 2.2-5: Antecedent Final Model PMF Hydrographs for P1, P2, P3, and P4



Missouri River at Brownville, NE

Figure 2.2-6: Subsequent Storm Final Model PMF Hydrographs for P1, P2, P3, and P4



Missouri River at Brownville, NE

Figure 2.2-7: Missouri River Junctions

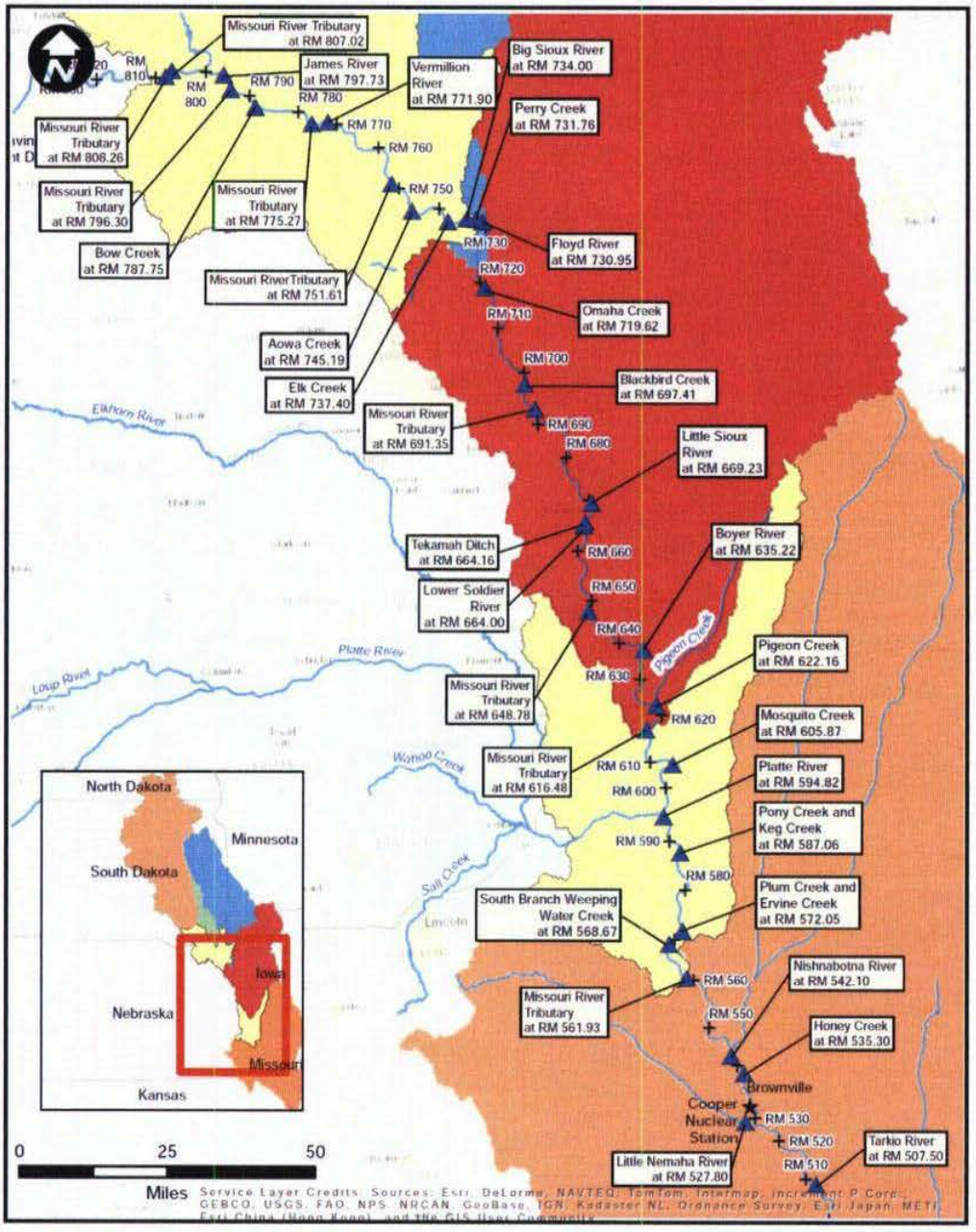


Figure 2.2-8: Full Floodplain PMF Hydrographs for Gavins Point Release of 160,000 cfs

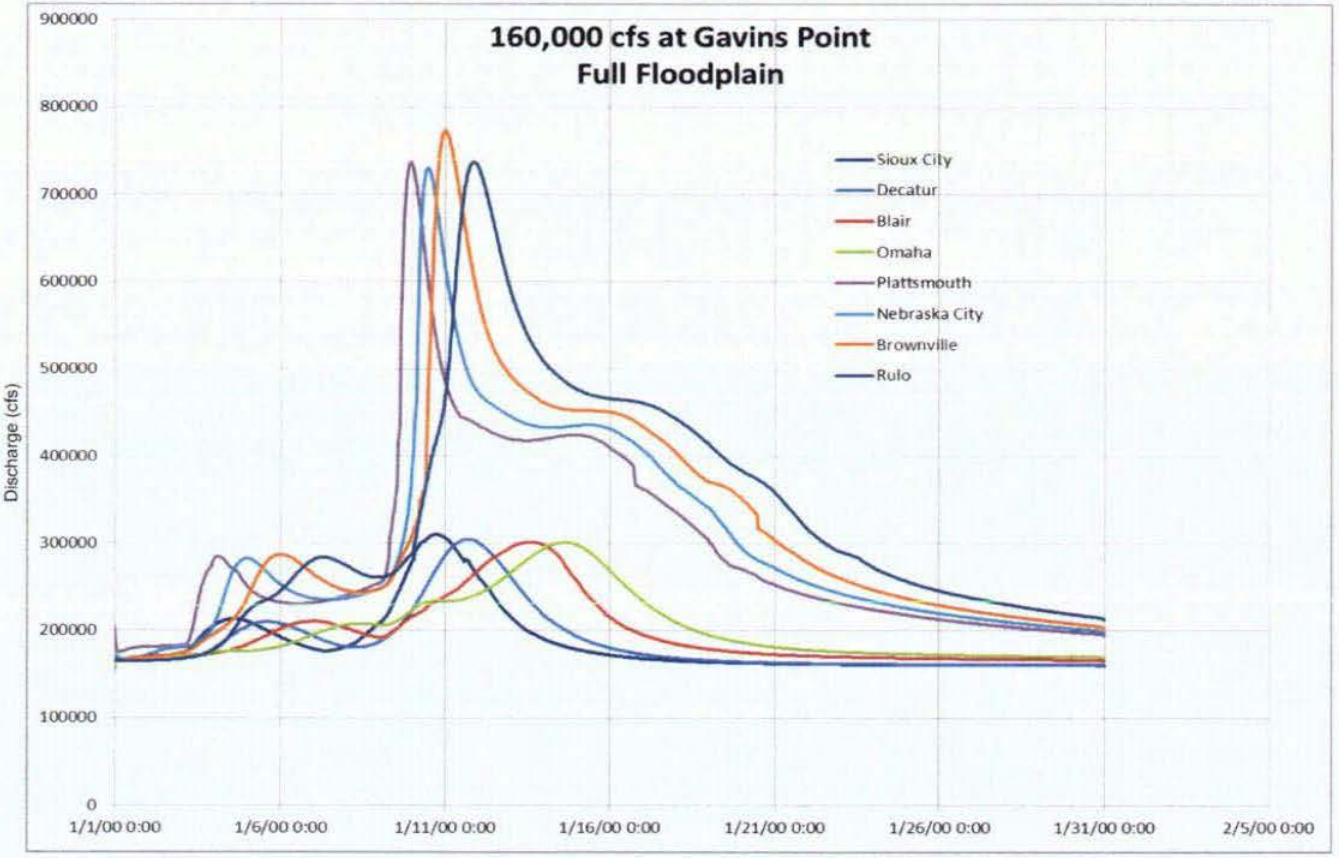


Figure 2.2-9: PMF Hydrographs for 2-D Model Upstream Boundary RM 556.16

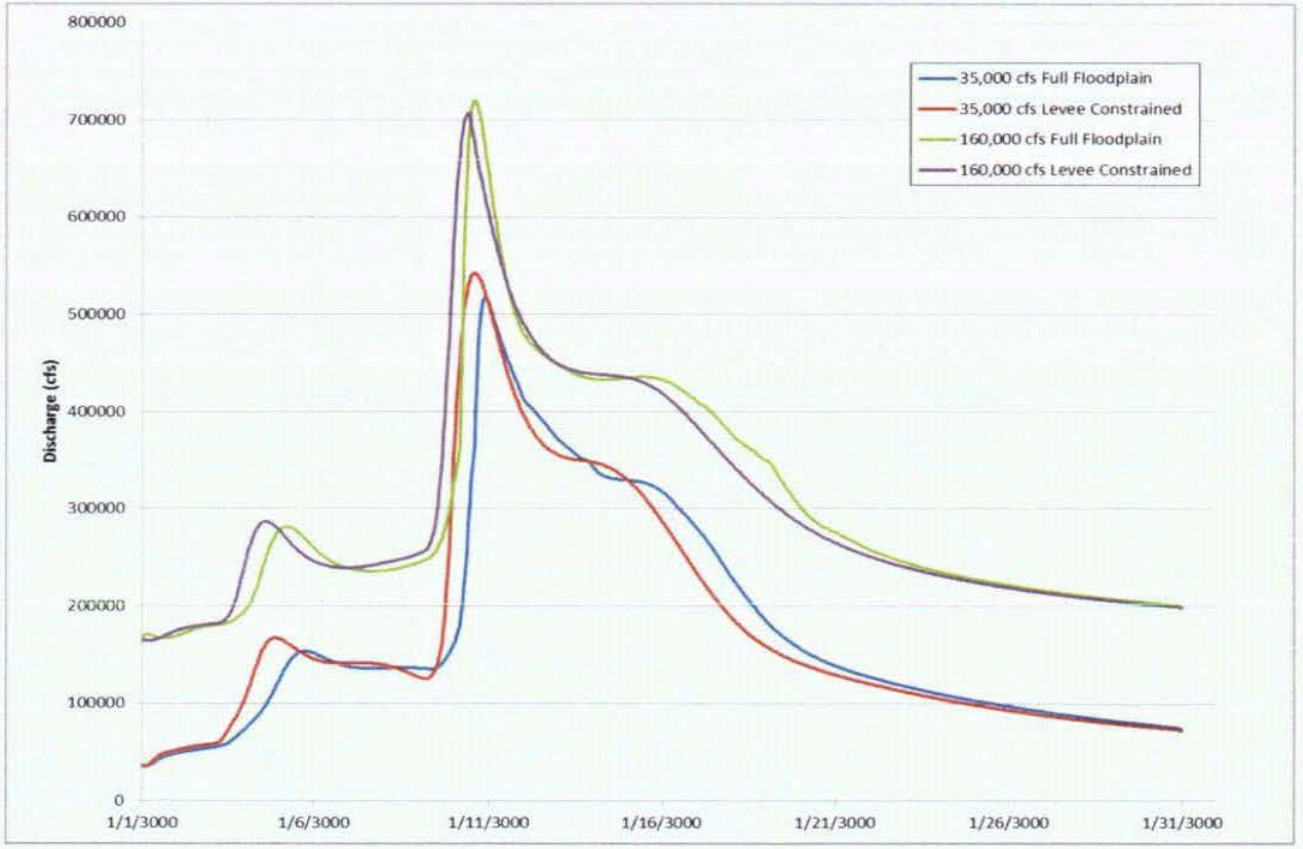


Figure 2.2-10: PMF Downstream Boundary Condition Rating Curve RM 510.03

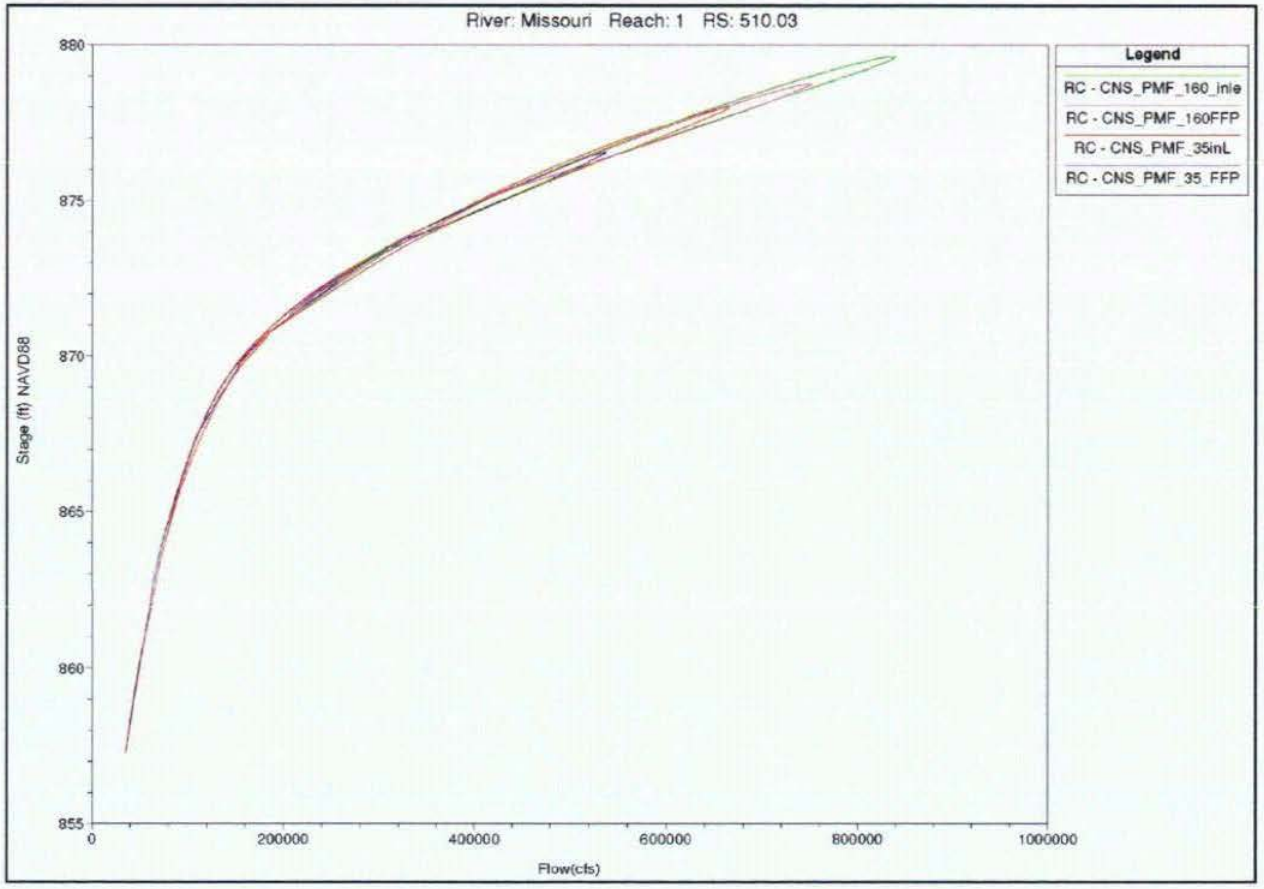


Figure 2.2-11: Study Area Overview for the 2-D Model

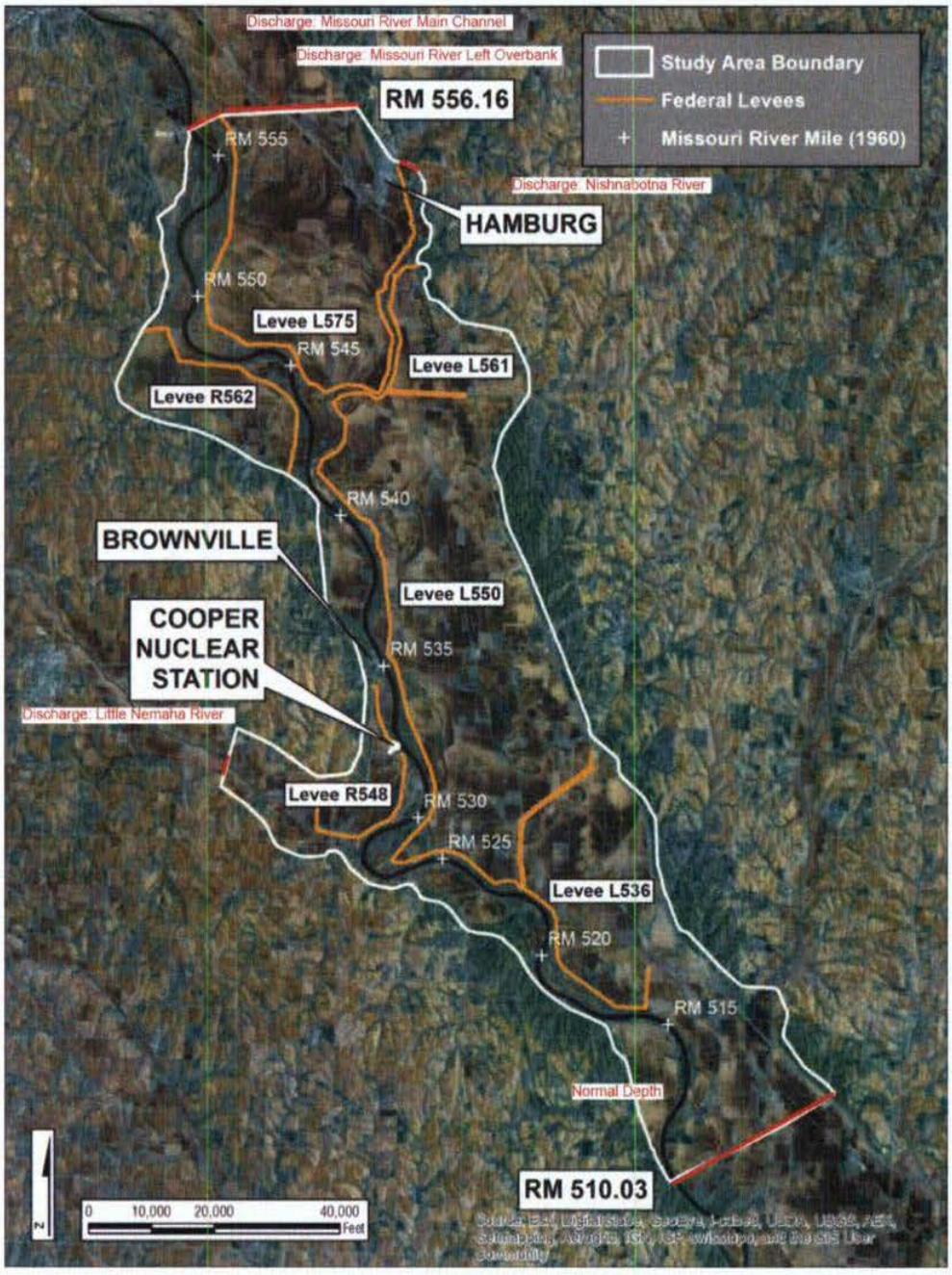


Figure 2.2-12: Spatial Distribution of Land Use Classes to Determine Manning's Roughness Coefficients

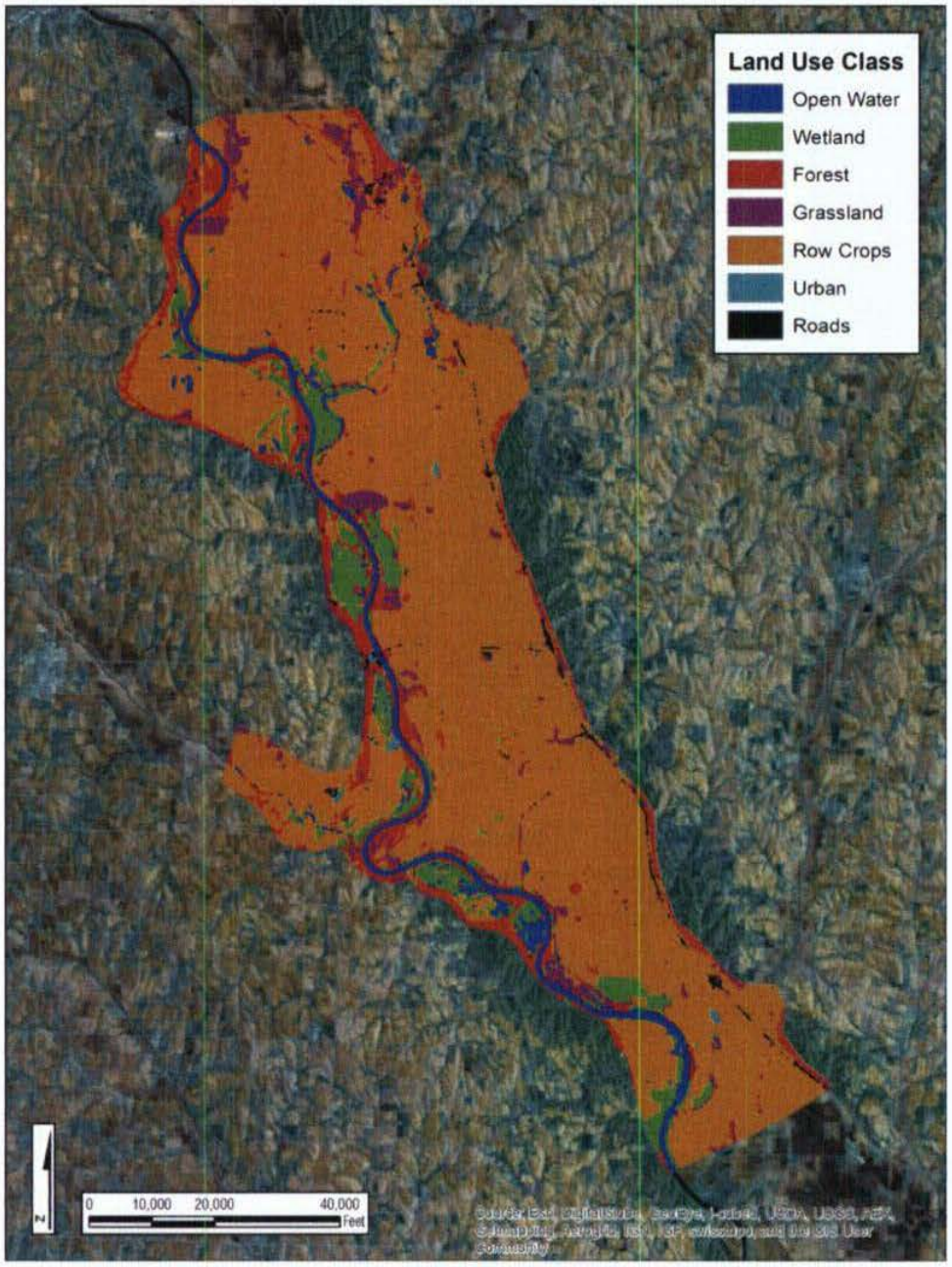


Figure 2.2-13: Computational Mesh Overview

Figure 2.2-14: Computational Mesh Near CNS

Figure 2.2-15: 2-D Model Inflow Hydrographs

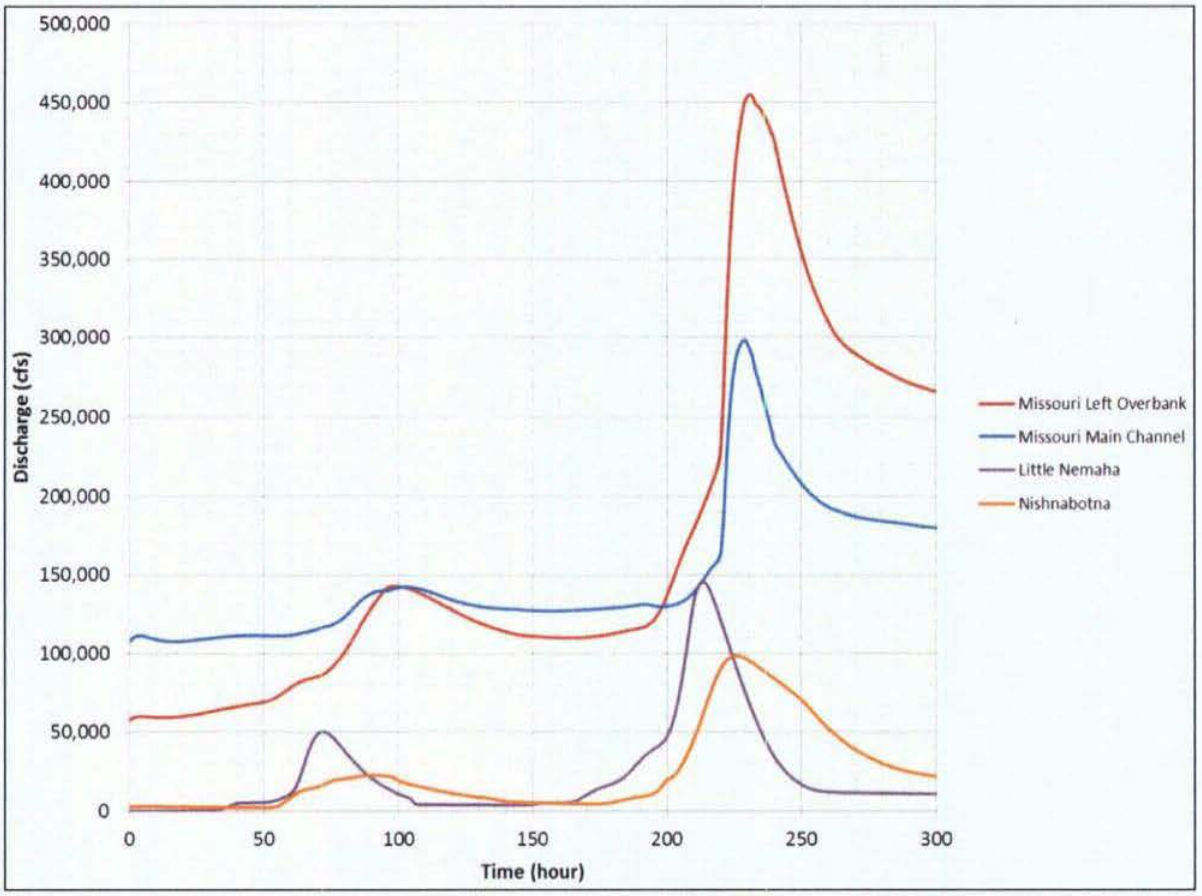


Figure 2.2-16: Comparison of Discharge Hydrographs through the Study Reach

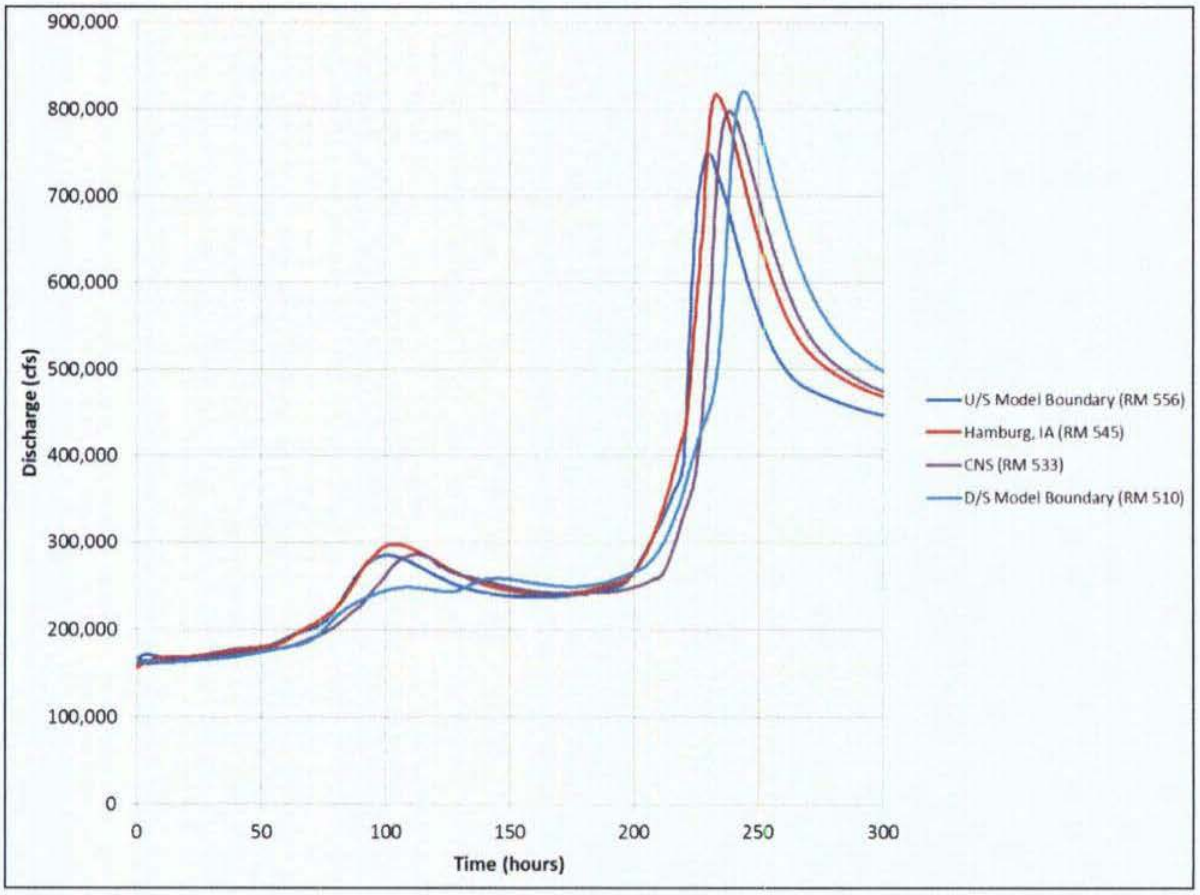


Figure 2.2-17: Contours of Maximum Water Surface Elevation for PMF Event – Large Scale

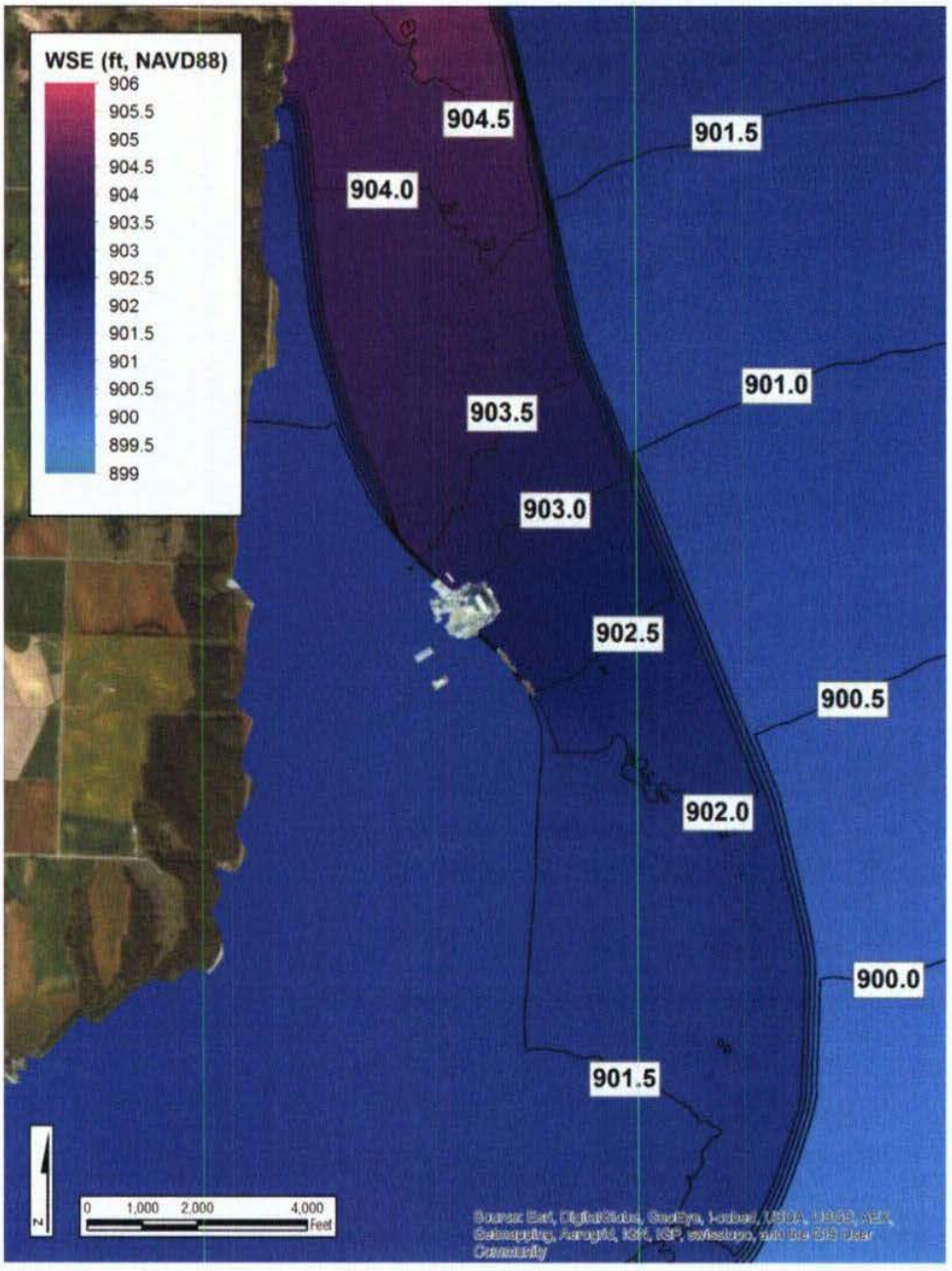


Figure 2.2-18: Contours of Maximum Water Surface Elevation for PMF Event – Small Scale

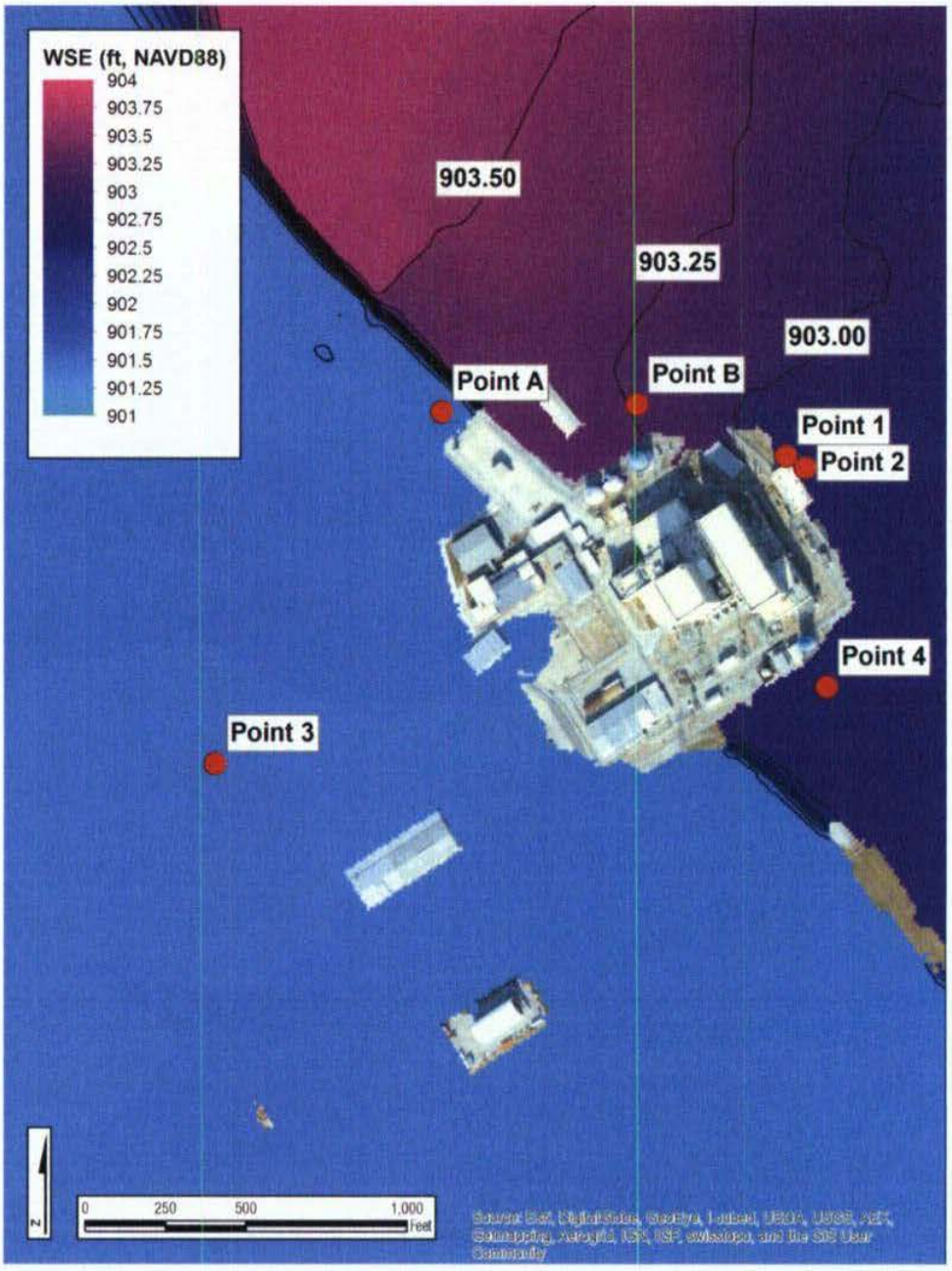


Figure 2.2-19: Contours of Maximum Depth for PMF Event - Overview

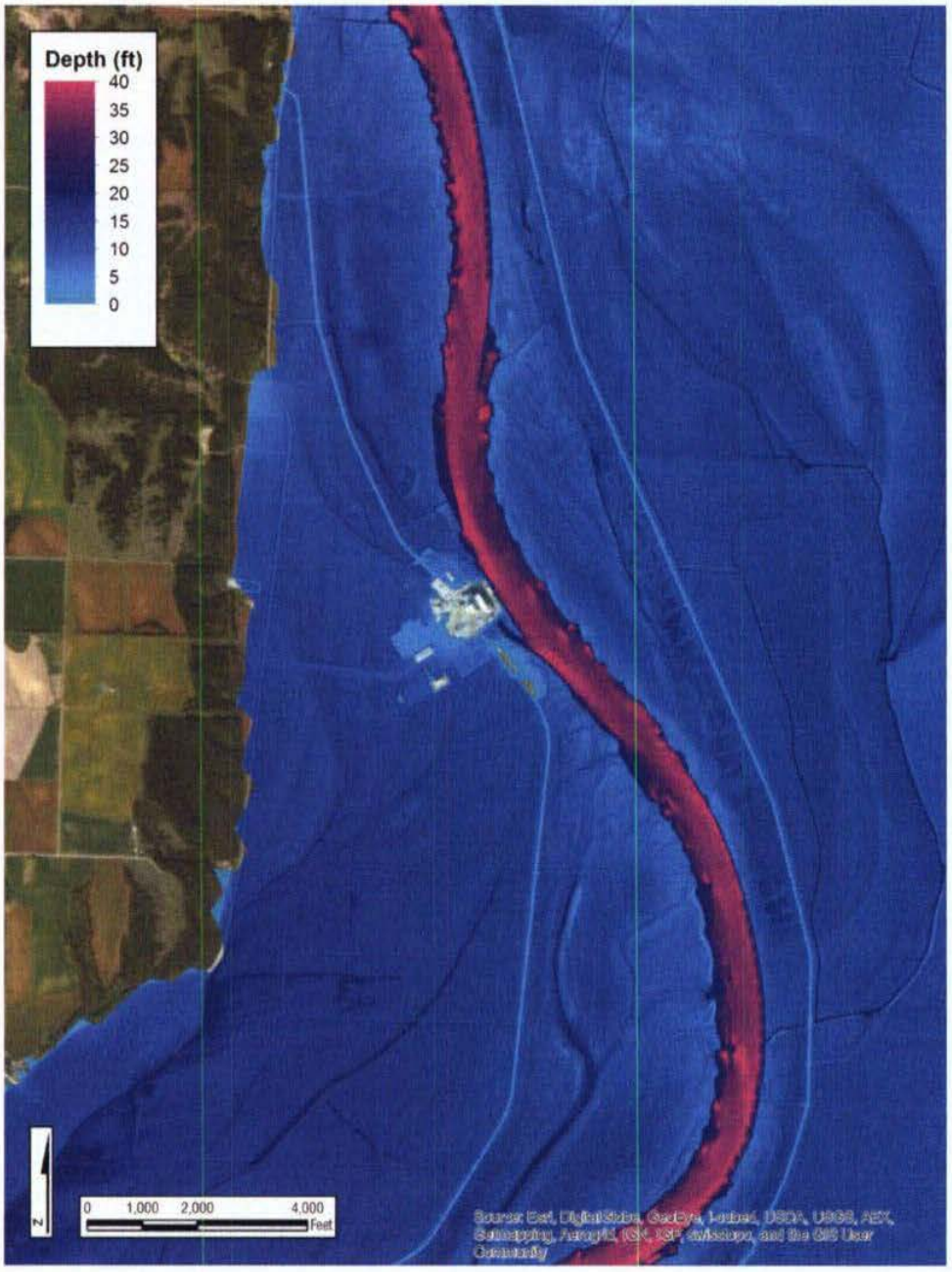


Figure 2.2-20: Contours of Maximum Depth for PMF Event – Small Scale

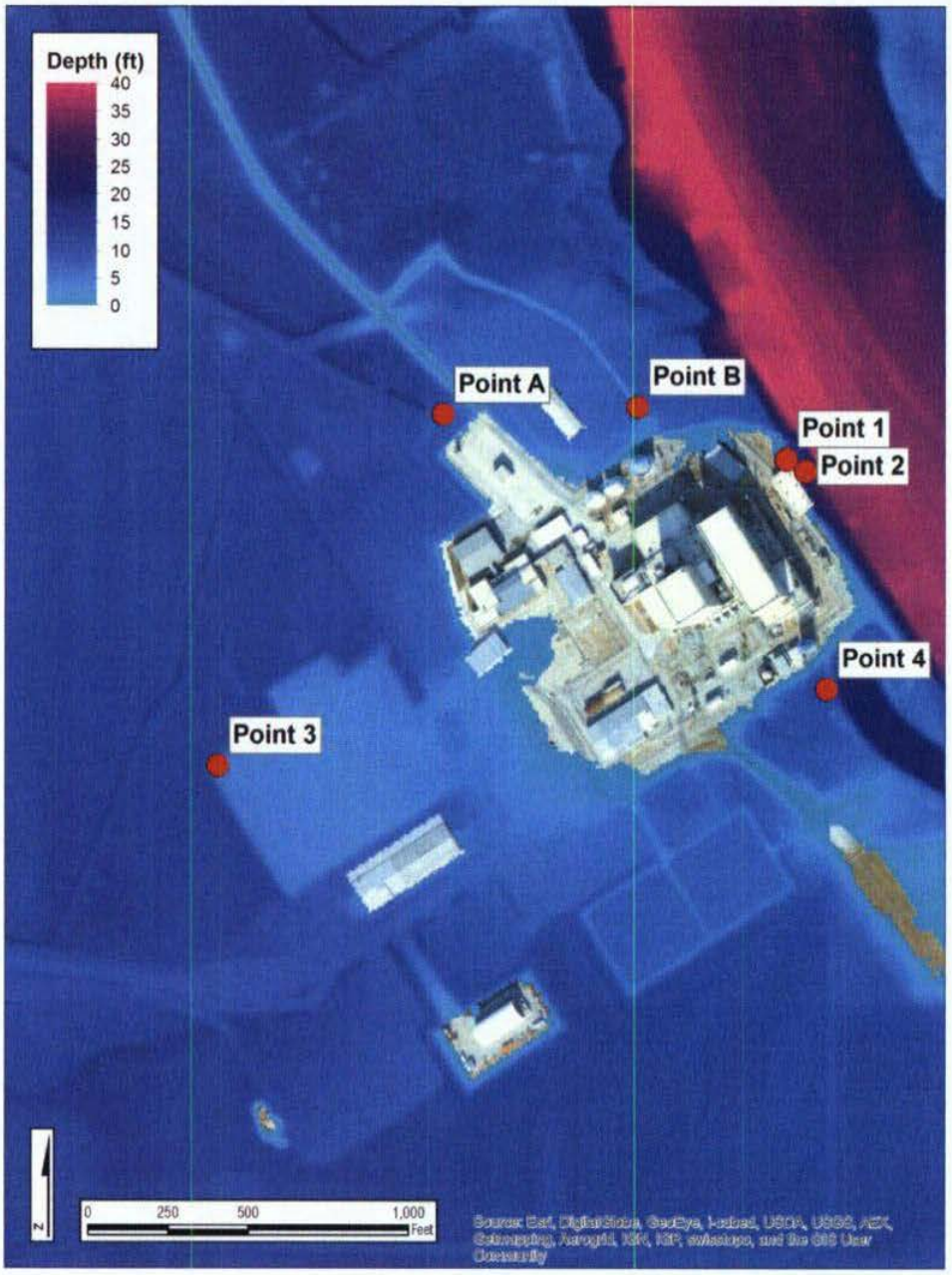


Figure 2.2-21: Contours of Maximum Velocity for PMF Event – Small Scale

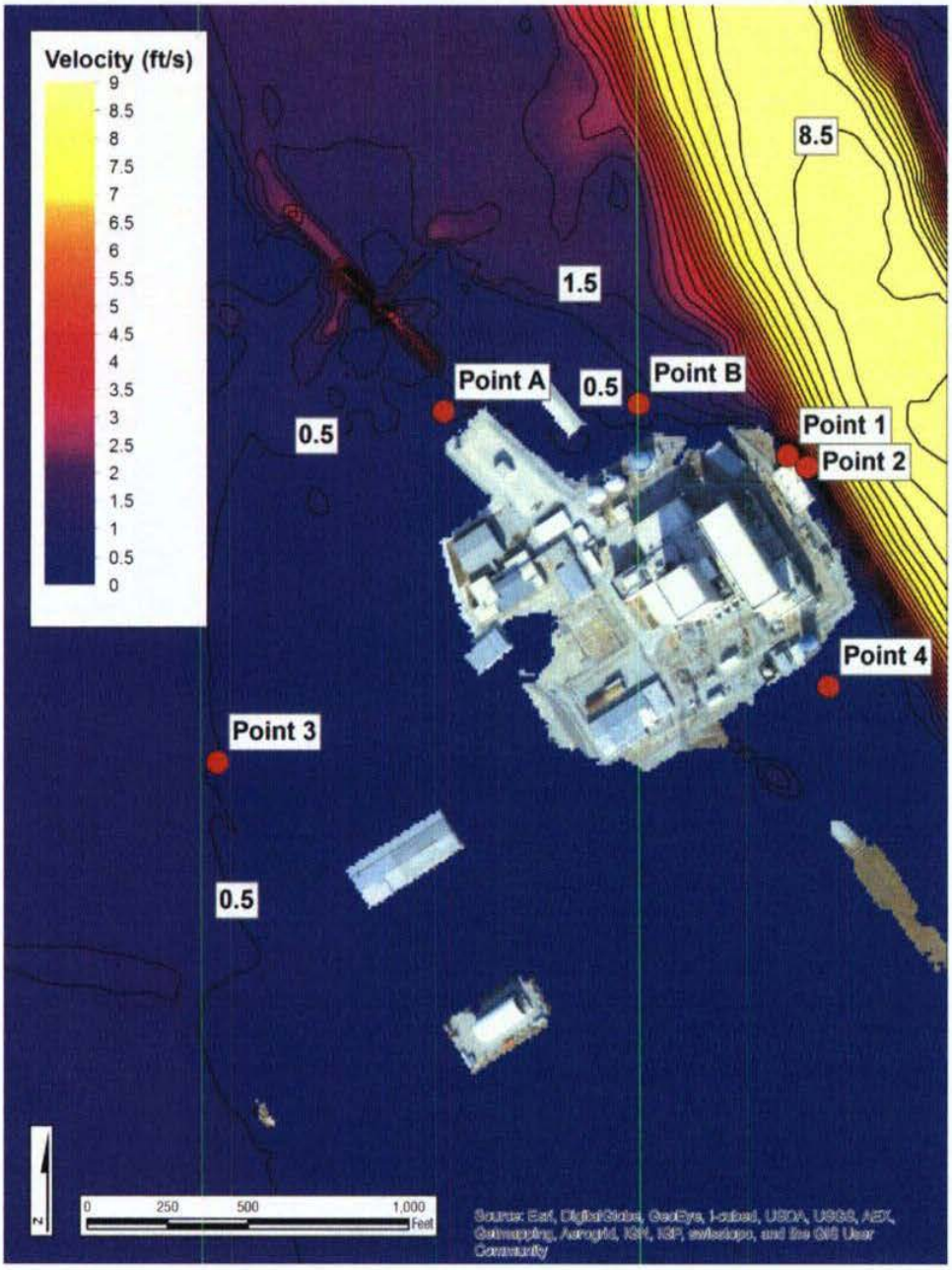


Figure 2.2-22: Fetches from Embankment Points of Interest

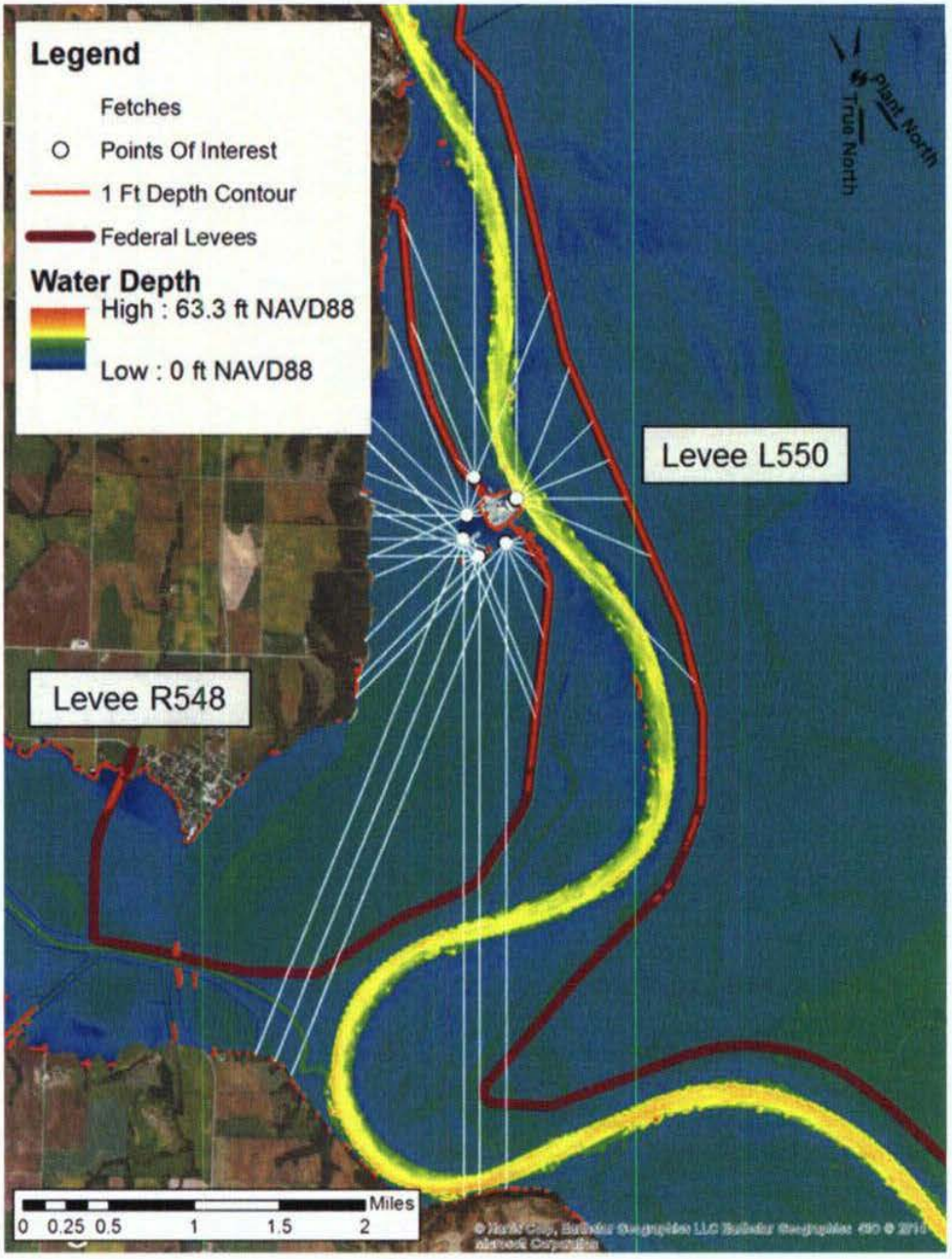
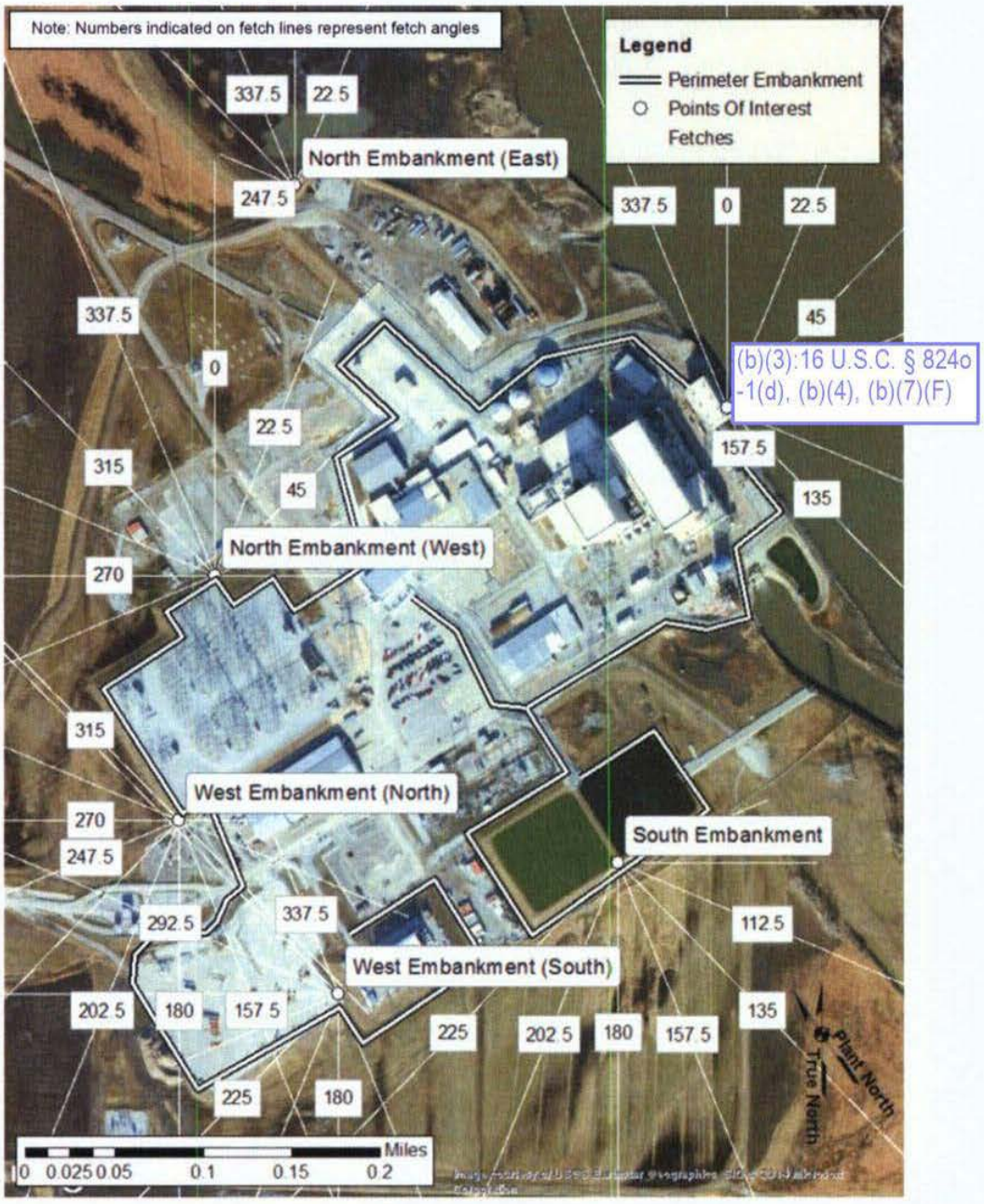


Figure 2.2-23: Labeled Points of Interest



(b)(3):16 U.S.C. § 824o -1(d), (b)(4), (b)(7)(F)

Figure 2.2-24: (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **and Locations of Interior Runup Calculations**

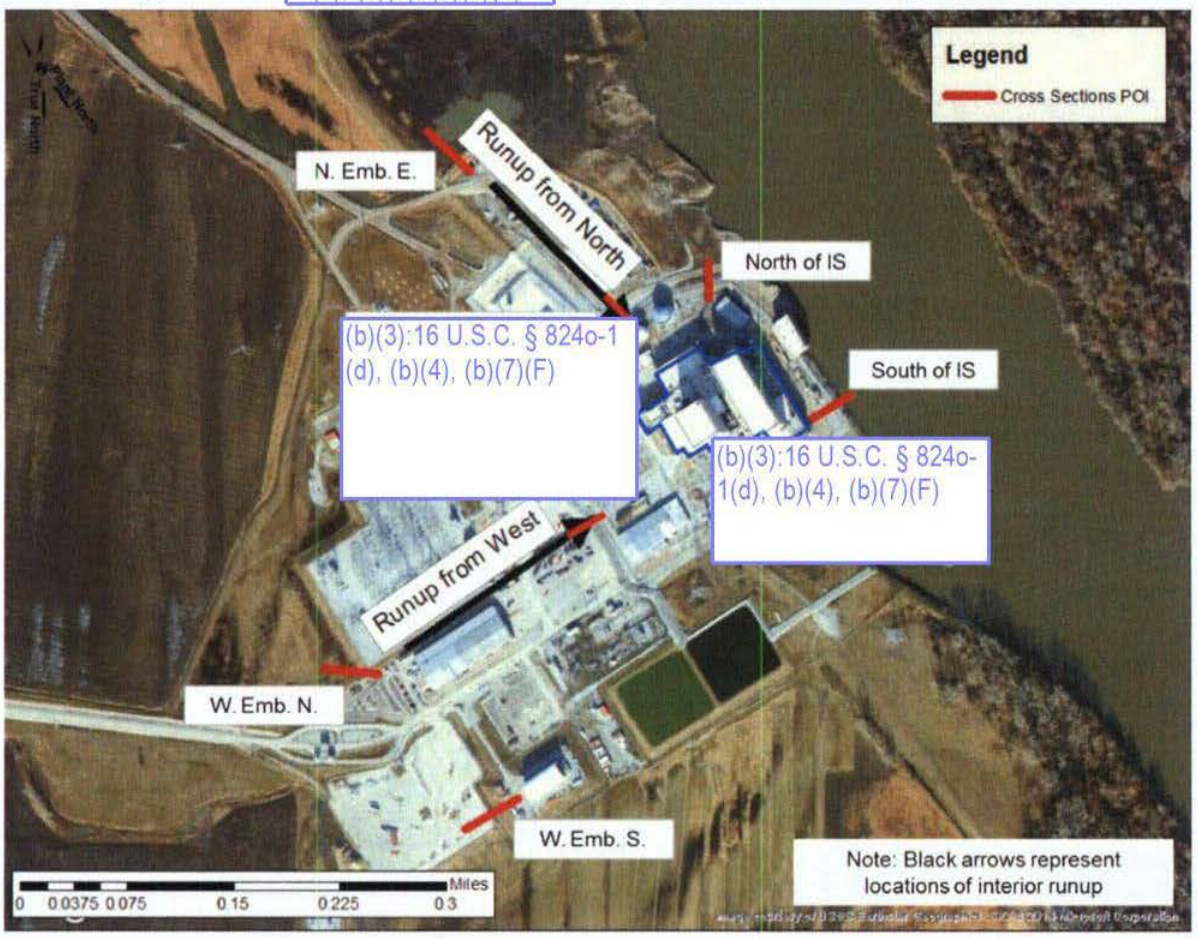


Figure 2.2-25: Expected Maximum Extent of Wave Runup at Representative Locations

2.3 DAM BREACHES AND FAILURES

The flooding reevaluation for Cooper Nuclear Station (CNS) adopted the approach and methodology for estimating the flood wave generated by upstream dam failure scenarios using the most current data available and the industry standard numerical modeling tool, Hydrologic Engineering Center–River Analysis System (HEC-RAS), Reference 2.3-1 and Hydrologic Engineering Center–Hydrologic Modeling System (HEC-HMS), Reference 2.3-2, from the U.S. Army Corps of Engineers (USACE). In particular, CNS analysis follows the guidance of ANSI/ANS 2.8-1992 (Reference 2.3-3) for the assessment of the potential dam failure modes and the specification of the antecedent and combined event conditions, consistent with the recommendations of NUREG/CR-7046 (Reference 2.3-4). The U.S. Nuclear Regulatory Commission (NRC) guidance on acceptable dam failure analysis methodologies is published as the Dam Failure Interim Staff Guidance (ISG) titled "Guidance for Assessment of Flooding Hazards Due to Dam Failure" (Reference 2.3-5), and was followed in this assessment. The methodology prescribed in the Dam Failure ISG document was the basis of the reevaluation for flooding effects at CNS due to upstream dam failures.

The complexity of the Missouri River hydrologic condition along with System and Non-System dams is discussed in Section 2.2 of this report. The locations of all System dams and all Non-System dams downstream of (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam, which is the System dam closest to CNS, are shown in Figure 2.3-1.

As discussed in Section 2.8.5 of this report, the Bank Stabilization and Navigation Program (BSNP) was authorized under various Congressional acts since 1912. Fort Peck Dam was authorized under the Rivers and Harbors Act of 1935. The lower five dams (Garrison, Oahe, Big Bend, Fort Randall, and Gavins Point) were authorized under the Flood Control Act of 1944. Additional bank stabilization projects were authorized by various Flood Control Acts. Further stream bank erosion controls were authorized under the various Water Resources Development Acts. The navigational channel project was declared officially finished in 1981, with the terminus of the project at RM 734.8 at Sioux City, Iowa. USACE actively maintains the channel. It is unknown whether USACE plans to revise channel maintenance procedures or the System dams.

Following the guidance of ANSI/ANS-2.8-1992 (Reference 2.3-3), the combined effects of wind setup and wave runup for a two-year design wind speed occurring coincidently with the maximum still water level as a result of the postulated upstream dam breach scenario were analyzed specifically for CNS Main Building facilities and Intake Structure.

As specified in the Dam Failure ISG , a large number of the dams in the watershed may have no impact on flooding at a site due to a combination of small dam size or large distance from the site. The Dam Failure ISG defines "inconsequential dams" as those dams that can be removed from consideration without analysis because they meet the criteria of having minimal or no adverse failure consequences beyond the dam owner's property. The Dam Failure ISG further suggests a second screening analysis for all dams that remain after removing the inconsequential dams, using several simplified modeling approaches for identifying dams whose failure would likely have negligible impacts on flooding at a nuclear power plant. Such dams are termed "noncritical dams." After the second screening, all remaining dams are considered "potentially critical dams," and detailed analyses are required to assess

these potentially critical dams to show which are truly critical to the flood hazard estimates at the CNS site. The potentially critical dams that are not shown to be critical through detailed analyses are assigned to the "noncritical dams" category. The Dam Failure ISG specifies that the cumulative effect of all noncritical dams should be carried forward and added to refined estimates for the critical dams. A screening-level analysis was performed to identify dams whose failure will have no impact to the CNS site.

The six System dams that are upstream of the CNS site on the mainstem of the Missouri River are critical dams. The salient features of these dams are presented in Table 2.3-1 from the Master Manual of Missouri River Mainstem Reservoirs (Reference 2.3-6). In addition to the information shown, more critical data related to each dam and its operations are required to perform the detailed upstream dam failures analysis for these System dams.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

These results were used to determine an overall dam failure flood hazard water surface elevation (WSEL) at the CNS site. The main steps followed for the upstream dam failures (combined System and Non-System dams) related to the CNS site are as follows:

- (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
-
-
-
-
-

- (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
-

Data used for the evaluation in this section include all dams located within the Missouri River Basin, downstream of Gavins Point Dam and upstream of the CNS site, as maintained in either federal or state inventories. The USACE National Inventory of Dams (NID) database (Reference 2.3-7) was queried for those states encompassed within the Missouri River Basin downstream of Gavins Point Dam and upstream of the CNS site. The states within the study area are Colorado, Iowa, Minnesota, Nebraska, North Dakota, South Dakota, and Wyoming. The NID database does not publish dam hazard classification; dam inventories maintained by each state's respective dam safety agency or state engineer's office contain these hazard classifications. Therefore, additional dam inventory data were directly obtained from relevant state agencies.

The 3,327 remaining dams after screening the inconsequential dams from the total Non-System dam list within the Missouri River Basin downstream of Gavins Point Dam and upstream of the CNS site are shown in Figure 2.3-2. In accordance with NUREG/CR-7046 (Reference 2.3-4) and Dam Failure ISG (Reference 2.3-5), the effort for evaluating the flood levels occurring due to dam failure was reduced by grouping dams together and representing them as hypothetical dams. Altogether, 49 hypothetical dams on different tributaries entering the Missouri River or the Platte River were considered. Dam break parameters of each hypothetical dam are presented in Table 2.3-2.

2.3.1 Flood Protection Level at the CNS Site

Maintenance Procedure 7.0.11 of CNS Operations Manual (Reference 2.3-8) reports that flood barriers are designed for a river flood water elevation of 906.00 ft CNS Plant Datum (906.37 ft NAVD88).The conversion from CNS plant datum to NAVD88 datum is provided in Reference 2.3-9.

2.3.2 Hydrologic Evaluation

2.3.2.1 HEC-HMS Model

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.2.2 Flow Hydrographs for Non-System Dams

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.2.3 Base Flow Data

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.2.4 USACE Dam Failure Hydrographs

The NRC transmitted the USACE analyses results for all six System dams for hydrologic and sunny-day dam failure. The USACE did not provide hydrographs resulting from seismic failure of the System dams because it concluded that the dams are seismically robust and not subject to failure during a seismic event. The USACE did not provide a description of its approach or assumptions used; however, the NRC confirmed that the detailed analyses performed by the USACE were in compliance with the Dam Failure ISG requirements. The USACE analysis of System dams included failure of Non-System dams upstream of Gavins Point Dam. The USACE provided hydrographs resulting from failures of the six System dams at a location immediately downstream of Gavins Point Dam. The USACE discharge hydrographs directly downstream of Gavins Point Dam were used for the combined System and Non-System dam failure analyses. Seven dam failure scenarios were selected for the combined System and Non-System dam failure analysis. These selections were based on bounding values of peak flow and timing to peak flow at the CNS plant site. The seven selected scenarios are listed below.

1. [redacted] *Hydrologic Failure*: (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) This scenario was selected because of its high magnitude of peak flow. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
2. [redacted] *Sunny-Day Failure*: This scenario was selected because of its high magnitude of peak flow. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
3. [redacted] *Hydrologic Failure:* [redacted] is the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The hydrologic failure scenario was selected because of its high magnitude of peak flow. USACE results indicated that the [redacted] sunny-day dam failure does not result in failure of [redacted] Dam downstream, so the [redacted] sunny-day dam failure scenario is bounded by other events and was not selected for detailed analysis. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
4. [redacted] *Hydrologic Failure*: This scenario was selected because [redacted] Dam is the closest upstream dam to [redacted] Dam and would result in the shortest time to peak at Gavins Point Dam of the upstream hydrologic failure scenarios. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
5. [redacted] *Sunny-Day Failure*: This scenario was selected because Fort Randall Dam is the closest upstream dam to [redacted] Dam and would result in the shortest time to peak at [redacted] Dam of the upstream sunny-day failure scenarios.
6. [redacted] *Hydrologic Failure*: [redacted] is the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7) The hydrologic failure scenario was selected because of its high magnitude of peak flow. USACE results indicated that the [redacted] sunny-day dam failure does not result in failure of [redacted] Dam (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) downstream, so the [redacted] sunny-day dam failure scenario is bounded by other events and was not selected for detailed analysis.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) 7. [redacted] *Hydrologic Failure*: While [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) this scenario was selected to show a comparison between peak flow and time to peak with other scenarios at the CNS site.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) None of the dam failure hydrographs for [redacted] Dam were considered. The failure results for this dam are not bounding in terms of flow magnitude or time to peak at the CNS site.

Hydrographs resulting from the seven selected dam failure scenarios listed above were used as the upstream flow boundary immediately downstream of [redacted] Dam ([redacted]) for seven different plans of the HEC-RAS model. The dam failure hydrographs at [redacted] Dam due to both hydrologic and sunny-day failures of the System dams are presented in Figure 2.3-5 and Figure 2.3-6, respectively. The seven selected System dam failure hydrographs downstream of [redacted] Dam used for the combined System dam and Non-System dam failure analysis are presented in Figure 2.3-7. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.3 Hydraulic Evaluation

A one-dimensional (1-D) HEC-RAS model of the Missouri River between [redacted] Dam (approximate [redacted]) and Rulo, Nebraska (approximate RM 498) was developed for the PMF for the CNS site, and it was used to route the dam breach hydrographs and compute the peak WSEL at the CNS site due to upstream dam failures. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.3.1 Route Dam Failure Hydrographs for Non-System Dams to CNS Site

The HEC-RAS model with the full floodplain geometry was used for upstream dam failure evaluation. Similar assumptions made for the model geometry for PMF were used. A summary of unsteady HEC-RAS model computational parameters are presented in Table 2.2-3 and Table 2.2-5. Initially, the Non-System dam failure analysis (peak flow estimated as 1.3 million cfs) was performed with no changes in the HEC-RAS model geometry. However, considering significant increases in the peak flow and volume of the System dam failure hydrographs ([redacted] cfs) compared to PMF runs, minor modifications to the floodplain geometry were made to obtain model stability without compromising model conservatism for the Non-System dam failure analysis. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Hydraulic Table Parameter (HTab) increments were increased by 10% and the number of points set to 100 at all cross sections to prevent the maximum discharge from exceeding the range of cross section HTabs. These HTab vertical increments ranged from 1.0 ft to 1.25 ft in the PMF HEC-RAS model, which ranges from 1.1 ft to 1.375 ft for dam break flood evaluation. All bridge features and cross sections upstream and downstream of the bridges were removed, which prevented water from being stored behind embankments, thus resulting in less attenuation and faster translation of hydrographs. Evaluating the upper end of peak dam failure flows showed that removing bridges and cross sections (when compared to simulations with bridges and sections included) did not have an effect on the peak WSEL or time to peak; however, this did improve model stability at bridge structure locations.

Sensitivity analyses were performed for the Non-System dam failure HEC-RAS model with both sets of geometries (with and without the changes described in paragraphs above) and found that the resulting WSELs differed only by a few tenths of a foot. For consistency, the same geometry with the above-described changes was used for Non-System dam failure analysis and combined System dam and Non-System dam failure analysis.

According to guidance from the Dam Failure ISG (Reference 2.3-5), levees that provide flood protection for the plant site should be considered to fail when overtopped. Although there is no levee that protects the CNS site, there are levees on the left and right banks of the Missouri River in the vicinity of the site. Consistent with this guidance for levees that protect the site, levees adjacent to the site would be considered to fail once overtopped. A full floodplain flow for the Non-System dam failure condition was considered as the WSEL would be near top of levee at the start of the simulation, which is the 500-year discharge in the Missouri River. Additional flows from the Non-System dam failure would provide enough discharge to overtop the levees early in the simulation. This consideration would have impact only on the rising limb of the dam breach hydrograph until the time that the levees overtop, and therefore, this condition was not expected to impact peak stage or discharge.

The 500-year flow from the USACE UMRSFFS report (Reference 2.3-11) was considered as the dam release from Gavins Point Dam for the Non-System dam failure analysis. The locations and peak inflows of the combined Non-System dam failure and 500-year flow hydrographs used in the model are summarized in Table 2.3-6. The peak computed WSEL at the CNS site was checked against the plant flood protection level to identify any potentially critical dams that require evaluation with detailed methods.

Based on the Dam Failure ISG, as discussed in Section 2.3 of this report, the peak WSEL was calculated. In accordance with the Dam Failure ISG, the cumulative effects of the noncritical dams were carried forward and added to refined estimates for the critical dams. The stage and discharge hydrographs at the CNS site and the comparison between the peak of the stage hydrograph and the design flood protection level at the CNS site are presented in Figure 2.3-8. The comparison of peak stage from hydrograph with CNS design flood protection level is presented in Table 2.3-7. The combined hypothetical dam failure hydrographs produce a peak stage of 903.09 ft NAVD88 at the CNS site, which is less than the existing plant flood protection level of 906.37 ft NAVD88; therefore, all Non-System dams below Gavins Point Dam and above the CNS site were considered as noncritical.

All HEC-RAS model simulations were executed and finished with stable and converged results.

2.3.3.2 Route Combined System and Non-System Dam Failure Hydrographs to the CNS Site

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.3.3 Water Level Magnitude and Timing

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.3.4 Velocities

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.4 Combined Effects

Impact at the CNS site resulting from wind wave action during an upstream dam failure in the Missouri River was evaluated. The contribution of waves to flood levels resulting from associated wind wave for the Main Building Complex and the Intake Structure was evaluated. The Hydrologic Failure, determined to be the bounding scenario at the CNS site based on estimated peak flow and stage, was used for this evaluation. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

The elevation of the top of foundation mat at the Intake Structure is 852.5 ft Plant Datum (852.9 ft NAVD88) and was used for computation of wave runup and total water levels for the Intake Structure. The nominal site grade of 903.00 ft Plant Datum (903.37 ft NAVD88) was used for the computation of wave runup and total water levels at the plant buildings. The 2-year mean recurrence interval annual extreme-mile wind speed from ANSI/ANS-2.8-1992 (Reference 2.3-3) was used for the estimation of wind wave heights. The fetch geometry was determined by extending lines at directional (radial angle) increments of 22.5 degrees from a representative point at the CNS site to the extent of the river cross sections in all directions as shown in Figure 2.3-15. Wind-driven waves were calculated using the Wind Speed Adjustment and Wave Growth module of the Automated Coastal Engineering System (ACES) in the Coastal Engineering Design & Analysis System (CEDAS) Version 4.03 (Reference 2.3-14). The ACES analysis was completed for each fetch. A controlling fetch was determined for each cardinal direction relative to the Plant North direction based on the largest wave height to approach from each direction. The fetches used to determine each cardinal direction are identified in Figure 2.3-16 and Figure 2.3-17. Wind setup was calculated using Equation 4 of U.S. Bureau of Reclamation (USBR)

ACER Technical Memorandum No. 2 (Reference 2.3-15). The calculated wind setup was added to this base WSEL for subsequent wave runup computations. In accordance with EM 1110-2-1614, Equation 2-2 (Reference 2.3-16) and ANSI/ANS-2.8-1992 (Reference 2.3-3), the minimum of the $H_{1\%}$ (the average wave height of the highest 1% of waves) and breaker wave height (0.78 times the Significant Wave height [H_s]) was used as the controlling wave height. The Main Building Complex is composed of multiple buildings with various roof elevations, as shown in Figure 2.3-18. Wave runup when the wave crest is at the wall was calculated on the Intake Structure and Main Building Complex in accordance with the Goda pressure formula as described in CEM Table VI-5-53 (Reference 2.3-17). Individual calculations were performed on Plant North, Plant East, Plant South, and Plant West directions using the highest building elevation exposed to each direction to determine the maximum wind wave runup experienced.

(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.5 Associated Flooding Impacts

2.3.5.1 Erosion and Sedimentation

(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) hydrologic dam failure results were considered for the evaluation of flooding effects because it resulted in the highest peak discharge at the CNS site. The three hydraulic cross sections at the CNS site (RMs 532.65, 532.53, and 532.49) were analyzed, and data from the hydraulic cross sections RMs 532.49 and 532.65 were used for the evaluation of associated flooding effects at the site. Cross section RM 532.49 is located near the downstream end of the CNS site and cross section RM 532.65 is located just upstream of the Intake Structure at the CNS site. A closer view of CNS with the modeled cross sections and modeled channel bank locations is presented in Figure 2.3-19. The critical time period was determined to be a two-and–a-half-day window (approximately 1.25 days before the peak discharge and 1.25 days after the peak discharge).

The maximum right overbank velocity and total discharge versus time during the crucial time window is presented in Figure 2.3-20. The channel velocity and total discharge versus time at the channel/right overbank transition near the location of the Intake Structure is presented in Figure 2.3-13. A comparison of WSELs and total discharge versus time is presented in Figure 2.3-21. The maximum right overbank bed shear stress and total discharge versus time is presented in Figure 2.3-22. The right overbank maximum velocity is 4.5 fps (Figure 2.3-20 and Table 2.3-10) and occurred at cross section RM 532.49 on the rising limb of the hydrograph. Additionally, as shown in Figure 2.3-13 and Table 2.3-10, the highest velocity at the channel/right overbank transition, near the location of the Intake Structure (cross section RM 532.65), is 18.0 fps. Figure 2.3-21 shows the peak WSEL attained is

937 ft. Figure 2.3-22 shows the calculated maximum right overbank bed shear stress is 0.95 pounds per square foot (psf).

The permissible maximum velocities of common localized groundcover at the CNS site include turf (7 fps), concrete (>18 fps), and 1-inch gravel (5 fps), as listed in Reference 2.3-18. The maximum right overbank velocity is 4.5 fps, which is near the lower threshold for turf, below the threshold for concrete, and near the upper threshold for 1-inch gravel. This may lead to localized erosion in areas that have 1-inch gravel. After periods of long inundation (exceeding 10 hours), research has shown that the permissible velocity of average turf cover is reduced to approximately 2 fps (Reference 2.3-18). This may lead to localized erosion in areas outside of the protected area and switchyard due to the duration of the event. The maximum velocity of 18 fps for the channel, identified for the Intake Structure, is below that of the maximum permissible velocity for concrete (>18 fps). Riprap is placed on the banks of the river and around the Intake Structure. The maximum velocity of 18 fps at the Intake Structure is higher than the maximum permissible velocity of riprap (13 fps). The permissible bed shear stresses of common groundcover at the CNS site, which includes turf (2.1 psf) and concrete (12.5 psf), are greater than 0.95 psf. The permissible bed shear stress of 1-inch gravel (0.33 psf), also prevalent at the site, is less than 0.95 psf. The maximum channel velocity of 18.0 fps, taken as a conservative estimate of the velocities to which the Intake Structure would be subjected, is less than the permissible velocity (>18 fps) for concrete. Riprap is placed on the banks of the river and around the Intake Structure. The maximum velocity around the Intake Structure of 18 fps is higher than the permissible velocity (13 fps) for riprap. On this basis, there is potential for damage to riprap and concrete around the Intake Structure. Considering permissible velocities and bed shear stress values, it would be expected that 1-inch gravel and smaller material would be subject to erosion during the peak of the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) hydrologic dam failure event. This would impact the Switchyard.

The USACE did not provide specific information with respect to the flow and stage hydrograph at each System dam, nor did it provide the dam breach parameters or estimated quantity of available source sediment from the breach, including material deposited behind the dam. Therefore, the sediment transport evaluation for the Missouri River during a System dam failure was qualitatively characterized based on available literature from the 2011 flood. It was assumed that the dam breach flows from a sediment transport perspective would be analogous to the 2011 dam releases, though at a much larger scale. The dam breach flows, much like the 2011 releases, would not include significant runoff and sediment from tributary inflows, but would include mobilized reservoir sediments.

The 2011 flood originated upstream of the mainstem dams in eastern Montana and western North Dakota and South Dakota (Reference 2.3-19). The flood flows were conveyed through the mainstem dam system, resulting in record discharge releases. As flows were conveyed from upstream of Fort Peck Dam through Gavins Point Dam, some of the sediment was deposited within the mainstem dam system. The prolonged flooding on the lower Missouri River that lasted over three months was primarily from the Gavins Point Dam releases, which were essentially sediment-free (Reference 2.3-20).

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.5.2 Inundation

Section 9.4 of the Dam Failure ISG (Reference 2.3-5) recommends developing inundation maps to provide assistance in identifying Systems, Structures, and Components (SSCs) important to safety that may require protective and/or mitigation measures from flooding due to dam breach. HEC-RAS 4.1 is equipped with geospatial capabilities that assist in quickly analyzing model results and mapping inundation limits. A thirty-meter (30-m) DEM dataset was converted into a binary floating point raster format (.flt) to be used by the floodplain mapping tool in RAS Mapper in HEC-RAS Version 4.1. The hydrologic dam failure case was selected for inundation estimation because it resulted in the highest water surface elevation at the CNS site. This floodplain boundary shape file from RAS Mapper was brought into ArcGIS 10.2 (Reference 2.3-21) to create inundation maps at different scales to provide a comprehensive visualization.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

The Light Detection and Ranging (LiDAR) dataset did not cover the entire study area; therefore, the 30-m DEM dataset was used to develop the inundation limits. Boundaries of the inundation maps in the vicinity of the plant site were revised. Roof elevations were assigned to the building footprints that were verified using plant drawings. These elevations were merged with the surface raster created using LiDAR data and the DEM survey data.

Using the ArcGIS 10.2 tool (Reference 2.3-21), the maximum flooding depth grid was converted into a raster format. Figure 2.3-23 shows the inundation mapping at a large scale with aerial imagery in the background, and provides an overview of the extent of inundation for the Missouri River adjacent to and upstream of the CNS site. The medium-scale map showing ground contours and flood velocity information is presented in Figure 2.2-24. The medium-scale map also shows that the depth grids surrounding the plant site are in the range of ft of depth of water in the . The small-scale map with names of SSCs, elevations, and local water depths is presented in Figure 2.2-25. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) structures is presented in Table 2.3-13. As shown in the Table 2.3-13, at the Office Building, Diesel Generator Building, and Heating

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

and Fan Building, flood levels are [redacted] elevations. The flood levels at other buildings are [redacted] elevation even though there will be flooding inside the buildings.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.5.3 Hydrostatic and Hydrodynamic Forces

Hydrodynamic impacts at the CNS site resulting from wind and wave action, and from currents during an upstream dam failure in the Missouri River were evaluated. The contribution of waves to flood levels, associated wave loads (as applicable), and current loads (as applicable) for the Main Building Complex and the Intake Structure was evaluated.

The elevation of the top of the foundation mat at the Intake Structure is 852.5 ft Plant Datum (852.9 ft NAVD88) and was used for computation of hydrodynamic and hydrostatic forces for the Intake Structure. The nominal site grade of 903.0 ft Plant Datum (903.37 ft NAVD88) was used for the computation of hydrostatic and hydrodynamic forces on plant buildings. Wind wave setup, wind wave heights, and total water levels evaluated, as discussed in Section 2.3.4, were used for estimation of hydrostatic and hydrodynamic forces on the plant structures.

Wave-induced hydrodynamic pressures for when the wave crest is at the wall were calculated on the Intake Structure and Main Building Complex in accordance with the Goda pressure formula as described in CEM Table VI-5-53 (Reference 2.3-17). The wave runup extent on the structures was also estimated. Wave-induced hydrodynamic pressures when the wave trough is at the wall were calculated on the Intake Structure and Main Building Complex in accordance with the Sainflou pressure formula as described in Coastal Engineering Manual (CEM) Table VI-5-52 (Reference 2.3-17). In addition, hydrostatic pressures were calculated using the weight of water and the depth of submerged structure during the upstream dam failure. The Main Building Complex is composed of multiple buildings with various roof elevations, as shown in Figure 2.3-18. Individual calculations were performed on Plant North, Plant East, Plant South, and Plant West directions using the highest building elevation exposed to each direction to determine the maximum pressure experienced.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

2.3.5.4 Debris Impact Loads

A review of possible flood debris items was undertaken to develop a spectra of flood debris sources. The following items are considered to represent a range of debris sources available upstream of the CNS plant. Two debris sources and masses are based on ASCE 7-10 (Reference 2.3-23) recommendations and the barge mass is based on typical barge sizes on the Missouri River. Additional vehicle and marine vessel masses are based on manufacturer data available on the Internet.

- ASCE 7-10 miscellaneous debris - 1 kip
- Large natural debris (ice, trees) - 4 kips
- Large vehicles and boats (tug boat, bus, mobile home) - 40 kips
- Tank-type debris (train cars, chemical tanks, semi-trailers) - 100 kips
- Barge - 5,100 kips

Flood debris impact loads were calculated following the methodology presented in Chapter C5 of ASCE 7-10 (Reference 2.3-23), with the following considerations:

- Duration of impact load of 0.03 seconds is recommended based on Reference 2.3-23, Section C5.4.4, with the pulse shape taken as a half sine wave.
- Velocity of the debris was considered to be equal to the water velocity, which may differ for each debris type depending on the expected debris path.
- Importance factor value of 1.3 for Risk Category IV was used (Reference 2.3-5).
- Depth coefficient was selected from Reference 2.3-23.
- Orientation coefficient value of 0.80 was used, as recommended in Reference 2.3-23.
- Blockage coefficient from Table C5-3 of Reference 2.3-23, considering sheltering within 100 ft upstream was used.
- Dynamic load factor depends on the fundamental vibration period of the impacted structure, and the maximum value from Table C5-4 of Reference 2.3-23 was conservatively used.

Estimated debris impact loads due to dam failure flood for different debris were determined using the input values identified above and based on channel and overbank velocities and are presented above in Table 2.3-17. As shown in the table, a barge with the maximum channel velocity of 18.0 fps will result in a maximum impact load of 167,800 kips.

2.3.6 References

2.3-1. U.S. Army Corps of Engineers (USACE). January 2010. River Analysis System HEC-RAS Version 4.1. USACE, Hydrologic Engineering Center, Davis, California.

2.3-2. U.S. Army Corps of Engineers (USACE). August 2010. Hydrologic Modeling System HEC-HMS Version 3.5. U.S. Army Corps of Engineers, Hydrologic Engineering Center: Davis, California.

2.3-3. American Nuclear Society. 1992. ANSI/ANS-2.8-1992. Determining Design Basis Flooding at Power Reactor Sites.

2.3-4. United States Nuclear Regulatory Commission (NRC). 2011. *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*. NUREG/CR-7046 PNNL-20091. U.S. Department of Energy, Office of Nuclear Regulatory Research: Richland, Washington.

2.3-5. United States Nuclear Regulatory Commission (NRC). July 29, 2013. Guidance for Assessment of Flooding Hazards Due to Dam Failure, Japan Lessons-Learned Project Directorate JLD-ISG-2013-01.

2.3-6. Missouri River, Mainstem Reservoir System, Reservoir Regulation Manual, Master Manual, U.S. Army Corps of Engineers, Omaha, Nebraska, 1979.

2.3-7. U.S. Army Corps of Engineers (USACE). National Inventory of Dams Database.

2.3-8. Nebraska Public Power District CNS Operations Manual, Maintenance Procedure 7.0.11, Rev. 29, "Flood Control Barriers."

2.3-9. Nebraska Public Power District Engineering Evaluation (EE) Number 12-035, Revision 0, "Review of 2012 Topographic Survey of CNS," March 2013.

2.3-10. United States Department of the Interior Bureau of Reclamation, ACER Technical Memorandum No. 11, December 1988: Downstream Hazard Classification Guidelines.

2.3-11. U.S. Army Corps of Engineers (USACE), November 2003, Upper Mississippi River System Flow Frequency Study, Hydraulics and Hydrology Appendix F, Missouri River, USACE Omaha District, Omaha, NE.

2.3-12. United States Geological Survey (USGS). 2013. USGS National Water Information System (NWIS). Washington, D.C., http://waterdata.usgs.gov/nwis/.

2.3-13. U.S. Army Corps of Engineers (USACE). January 2010. HEC-RAS River Analysis System, Hydraulic Reference Manual. USACE Hydrologic Engineering Center, Davis, CA.

2.3-14. CEDAS-ACES Version 4.03 (2014). Veri-Tech, Vicksburg, MS.

2.3-15. United States Bureau of Reclamation. 1981. Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams. ACER Technical Memorandum No. 2.

2.3-16. U.S. Army Corps of Engineers (USACE). 1995. Engineer Manual 1110-2-1614. Design of Coastal Revetments, Seawalls, and Bulkheads.

2.3-17. U.S. Army Corps of Engineers (USACE). 2008. Coastal Engineering Manual. EM 1110-2-1100, Washington, D.C. (in 6 volumes).

2.3-18. Fischenich, C. May 2001. *Stability Thresholds for Stream Restoration Materials.* Ecosystem Management and Restoration Research Program, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

2.3-19. United States Department of Commerce. May 2012. The Missouri/Souris River Floods of May – August 2011. Prepared by the National Oceanic and Atmospheric Administration, National Weather Service, Kansas City, MO, and Salt Lake City, UT.

2.3-20. United States Geological Survey 2013a. "Characteristics of Sediment Transport at Selected Sites along the Missouri River during the High-Flow Conditions of 2011", U.S. Geological Survey Paper 1798-F, 27p. http://pubs.usgs.gov/sir/2013/5006/sir13-5006.pdf.

2.3-21. ArcMap Version 10.2 (2013). ESRI, Redlands, CA.

2.3-22. Dean, R. and Dalrymple, R. 1991. Water Wave Mechanics for Engineers and Scientists. World Scientific. Hackensack, NJ.

2.3-23. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures."

2.3.7 Tables

Tables associated with Section 2.3 are presented on the following pages.

Table 2.3-1: Salient Features of Missouri River Mainstem Dams

Parameter	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
River Mile	
Top of dam elevation (ft MSL)	
Spillway crest elevation (ft MSL)	
Maximum operating pool elevation (ft MSL)	
Maximum normal operating pool elevation (ft MSL)	
Base flood control elevation (ft MSL)	
Minimum operating pool elevation (ft MSL)	
Gross storage elevation range (ft MSL)	
Gross storage (ac-ft)	

Table 2.3-2: Dam Break Parameters for Hypothetical Dams

River/Creek	Max. Storage (ac-ft)	Dam Crest Elevation (ft NAVD88)	Dam Breach Bottom Elevation (ft NAVD88)	Dam Height (ft)	Breach Bottom Width (ft) USBR Equation	Breach Formation Time (hrs) USBR Equation	Peak Breach Outflow (cfs)
James u/s Dam	605,585	1,355.7	1,295.7	60.0	180.0	0.60	256,459
James d/s Dam	163,540	1,185.7	1,151.7	34.0	102.0	0.34	62,108
Beaver Creek Dam	1,167	1,202.7	1,179.7	23.0	69.0	0.23	21,789
Antelope Creek Dam	3,142	1,218.1	1,179.2	38.9	117.0	0.39	73,644
Bow Creek Dam	375	1,150.6	1,140.7	9.9	29.7	0.10	2,816
Vermillion Creek Dam	3,160	1,148.7	1,126.7	22.0	66.0	0.22	20,709
Aowa Creek Dam	15,774	1,147.4	1,100.6	46.8	140.0	0.47	131,832
Elk Creek Dam	53	1,263.5	1,236.5	27.0	81.0	0.27	4,581
Big Sioux Dam	12,030	1,135.3	1,096.7	38.6	116.0	0.39	81,841
Perry Creek Dam	1,211	1,114.3	1,099.7	14.6	44.0	0.15	7,444
Floyd River Dam	4,952	1,107	1,086.9	20.1	60.0	0.20	16,560
Bacon Creek Dam	2,934	1,161.1	1,118.6	42.5	128.0	0.43	82,505
Pigeon Creek Ditch Dam	7,654	1,186.4	1,136.2	50.2	151.0	0.50	142,379
Omaha Creek Dam	238	1,102.5	1,085.5	17.0	51.0	0.17	9,582

Table 2.3-2: Dam Break Parameters for Hypothetical Dams, continued

River/Creek	Max. Storage (ac-ft)	Dam Crest Elevation (ft NAVD88)	Dam Breach Bottom Elevation (ft NAVD88)	Dam Height (ft)	Breach Bottom Width (ft) USBR Equation	Breach Formation Time (hrs) USBR Equation	Peak Breach Outflow (cfs)
Blackbird Creek Dam	125	1,066.0	1,052.1	13.9	42.0	0.14	5,718
Elm Creek Dam	204	1,108.6	1,063.6	45.0	135.0	0.45	110,983
Big Whisky Creek Dam	1,917	1,107.5	1,086.7	20.8	62.0	0.21	17,504
WF Little Sioux Dam	8,665	1,188.8	1,136.2	52.6	158.0	0.53	156,182
Wolf Creek Dam	9,971	1,145.7	1,080.7	65.0	195.0	0.65	229,603
Cottonwood Weber Creek Dam	1,607	1,129.6	1,097.6	32.0	96.0	0.32	44,397
Little Sioux River Dam	57,984	1,066.8	1,039.5	27.3	82.0	0.27	35,938
Maple Creek Dam	18,622	1,058.3	1,035.6	22.7	68.0	0.23	22,556
Tekamah Creek Dam	13,144	1,112.4	1,043.2	69.2	208.0	0.69	286,481
Silver Creek Dam	2,591	1,088.5	1,041.4	47.1	141.0	0.47	108,240
Soldier River Dam	5,542	1,042.3	1,030.6	11.7	35.0	0.12	4,305
Cameron Ditch Dam	981	1,071.8	1,045.5	26.3	79.0	0.26	29,244
Long Creek Dam	943	1,038.7	1,000.0	38.7	116.0	0.39	51,803
De Soto Bend Dam	45,400	1,005.5	980.5	25.0	75.0	0.25	28,722
Moores Creek Dam	411	1,039.7	1,005.4	34.3	103.0	0.34	24,614
Boyer River Dam	37,174	1,015.9	981.5	34.4	103.0	0.34	63,610
Deer Creek Dam	1,095	1,049.8	1,019.9	29.9	90.0	0.30	38,270

Table 2.3-2: Dam Break Parameters for Hypothetical Dams, continued

River/Creek	Max. Storage (ac-ft)	Dam Crest Elevation (ft NAVD88)	Dam Breach Bottom Elevation (ft NAVD88)	Dam Height (ft)	Breach Bottom Width (ft) USBR Equation	Breach Formation Time (hrs) USBR Equation	Peak Breach Outflow (cfs)
Pigeon Creek Dam	5,113	1,007.1	981.3	25.8	77.0	0.26	30,620
Mosquito Creek Dam	5,661	1,009.9	970.7	39.2	118.0	0.39	81,804
Papillion Creek Dam	103,252	10,16.9	950.4	66.5	200.0	0.67	321,533
Seminoe Dam	1,519,519	6,426.9	6,150.9	276.0	828.0	2.76	10,048,395
Pathfinder Dam	1,128,432	5,919.8	5,625.2	294.6	884.0	2.95	10,476,655
Glendo Dam	1,348,025	4,724.9	4,503.9	221	663.0	2.21	6,080,174
North Platte at Conf Dam	2,887,978	2,862.4	2,747.9	114.5	344.0	1.15	1,288,984
South Platte at Conf Dam	3,468,545	2,870.4	2,747.9	122.5	368.0	1.23	1,523,906
Platte D/S Dam	1,105,332	1,051.3	940.30	111	333.0	1.11	1,173,884
Plattsmouth Dam	365	996.4	968.5	27.9	84.0	0.28	24,120
Pony Keg Creek Dam	9,608	996.8	956.1	40.7	122.0	0.41	93,408
Rock Creek Dam	1,3454	1,030.4	938.0	92.4	277.0	0.92	303,052
Waubonsie Creek Dam	16,318	1,024.9	932.4	92.5	278.0	0.93	429,799
Rakes Creek Dam	671	969.1	939.4	29.7	89.0	0.30	30,909
Weeping Water Creek Dam	16138	965.3	924.6	40.7	122.0	0.41	94,837
Nebraska City Dam	74	955.5	918.5	37.0	111.0	0.37	50,699
Horse Creek Dam	210	967.3	942.6	24.7	74.0	0.25	17,231
Camp Creek Dam	285	952.5	908.5	44.0	132.0	0.44	66,847

Table 2.3-3: HEC-HMS Model Input Parameters and Definitions

Parameter	Definition
Muskingum-Cunge Channel Routing	
Channel length	~2000' – ~580,000'
Channel slope	Average slope for the whole channel (0.0001-0.004)
Channel Manning's roughness coefficient	Average Manning's roughness coefficient for the whole length of the main channel, exclusive of the overbank areas (0.02-0.065).
Channel shape	Shape of channel (eight-point cross section)
Channel width	Width of channel (~800' – ~8000')
Left bank Manning's roughness coefficient	Manning's roughness coefficient for the left overbank of the channel (0.05-0.085)
Right bank Manning's roughness coefficient	Manning's roughness coefficient for the right overbank of the channel (0.05-0.085)
Reservoir	
Routing method	Outflow Structures
Storage method	Elevation Storage curve using 10m DEM data
Initial storage	Pool elevation at dam height. (Assumes the reservoir is full)
Tailwater method	Provides the tailwater at the reservoir outflow. For this Calculation, no tailwater was assumed. (Assumes reservoir tailwater has no effect on the reservoir outflow.)
Time step control	Automatic Adaption (1-minute time step for control specification).

Table 2.3-3: HEC-HMS Model Input Parameters and Definitions, continued

Parameter	Definition
Dam Tops	Represents top of dam where water goes over the dam top in an uncontrolled manner.
Level dam top	Assumes flow over the dam can be represented as a broad-crested weir.
Crest elevation	Elevation of the dam top
Length of dam	Total crest length over which water passes
Discharge coefficient	Accounts for energy losses as water approaches the dam top and flows over the dam.
Dam Break	
Overtop dam break	Represents failures caused by overtopping of the dam.
Top elevation	Top of the dam face
Bottom elevation	Elevation of the bottom of the trapezoidal or rectangular opening in the dam face when the breach is fully developed.
Bottom width	Width of the bottom of the trapezoidal or rectangular opening in the dam face when the breach is fully developed.
Left and right side slope	Units or horizontal distance per one unit of vertical distance (0 for rectangular breach).
Development time	Total time for the breach to form from initiation to reaching the maximum breach size (in hours).
Trigger method: Elevation	Pool elevation when the failure starts. This is set slightly below the elevation of maximum storage.
Progression method	Determines how breach grows from initiation to maximum size during the development time. For linear method, breach grows in equal increments of depth and width.

Table 2.3-4: Hypothetical Dam Outflow Summary from HEC-HMS Model

Hypothetical Dam Name	Peak Breach Outflow at Reservoir Outlet (cfs)	River Mile (as in HEC-RAS model) of Lateral Inflow	Dam Failure Hydrograph Peak at Lateral Inflow Location (cfs)	Remarks
James u/s Dam	256,459	N/A	N/A	James U/S Dam flow is being routed down to Junction JJR1.
James d/s Dam	62,108	797.73	120,810	Routed flow from James u/s Dam and outflow from James d/s is being reported at Junction JJR1.
Beaver Creek Dam	21,789	806.23	21,789	
Antelope Creek Dam	73,644	803.85	73,644	
Bow Creek Dam	2,816	787.75	2,816	
Vermillion Creek Dam	20,709	771.9.0	20,709	
Aowa Creek Dam	131,832	745.19	131,832	
Elk Creek Dam	4,581	737.4.0	4,581	
Big Sioux Dam	81,841	734.00	81,841	
Perry Creek Dam	7,444	731.76	7,444	
Floyd River Dam	16,560	730.95	16,560	
Bacon Creek Dam	82,505	730.54	82,505	
Pigeon Creek Ditch Dam	142,379	720.03	142,379	
Omaha Creek Dam	9,582	719.62	9,582	
Blackbird Creek Dam	5,718	697.41	5,718	
Elm Creek Dam	110,983	690.95	110,983	
Big Whisky Creek Dam	17,504	669.83	400,997	Outflow from WF Little Sioux, Wolf Creek, Cottonwood, Weber Creek, and Big Whisky Creek being reported at Junction TUMI.

Table 2.3-4: Dam Outflow Summary from HEC-HMS Model, continued

Hypothetical Dam Name	Peak Breach Outflow at Reservoir Outlet (cfs)	River Mile (as in HEC-RAS model) of Lateral Inflow	Dam Failure Hydrograph Peak at Lateral Inflow Location (cfs)	Remarks
WF Little Sioux Dam	156,182	N/A	N/A	Outflow is being reported at Junction TUMI.
Wolf Creek Dam	229,603	N/A	N/A	Outflow is being reported at Junction TUMI.
Cottonwood Weber Creek Dam	44,397	N/A	N/A	Outflow is being reported at Junction TUMI.
Little Sioux River Dam	35,938	669.23	58,429	Outflow from Maple Creek and Little Sioux River being combined at Junction Little Sioux-Maple Junction.
Maple Creek Dam	22,556	N/A	N/A	Outflow is being reported at Junction Little Sioux-Maple.
Tekamah Creek Dam	286,481	664.56	297,824	Outflow from Silver Creek and Tekamah Creek being combined at Junction Tekamah-Silver.
Silver Creek Dam	108,240	N/A	N/A	Outflow is being reported at Junction Tekamah-Silver.
Soldier River Dam	4,305	664.00	4,305	
Cameron Ditch Dam	29,244	658.91	29,244	
Long Creek Dam	51,803	645.15	51,803	
De Soto Bend Dam	28,722	641.91	28,722	
Moores Creek Dam	24,614	640.73	24,614	
Boyer River Dam	63,610	635.22	63,610	
Deer Creek Dam	38,270	632.99	38,270	
Pigeon Creek Dam	30,620	622.16	30,620	
Mosquito Creek Dam	81,804	605.87	81,804	
Papillion Creek Dam	321,533	596.47	321,533	

Table 2.3-4: Dam Outflow Summary from HEC-HMS Model, continued

Hypothetical Dam Name	Peak Breach Outflow at Reservoir Outlet (cfs)	River Mile (as in HEC-RAS model) of Lateral Inflow	Dam Failure Hydrograph Peak at Lateral Inflow Location (cfs)	Remarks
Seminoe Dam	10,048,395	N/A	N/A	Seminoe Dam flow is being routed down to Junction Platte_DS_JN.
Pathfinder Dam	10,476,655	N/A	N/A	Pathfinder Dam flow is being routed down to Junction Platte_DS_JN.
Glendo Dam	6,080,174	N/A	N/A	Glendo Dam flow is being routed down to Junction Platte_DS_JN.
North Platte at Conf Dam	1,288,984	N/A	N/A	North Platte at conf Dam flow is being routed down to Junction Platte_DS_JN.
South Platte at Conf Dam	1,523,906	N/A	N/A	South Platte at conf Dam flow is being routed down to Junction Platte_DS_JN.
Platte D/S Dam	1,173,884	594.82	1,601,927	Routed flow from Seminoe, Glendo, Pathfinder, North and South Platte at conf Dams and outflow from Platte D/S is being reported at Junction Platte_DS_JN.
Plattsmouth Dam	24,120	594.14	24,120	
Pony Keg Creek Dam	93,408	587.06	93,408	
Rock Creek Dam	303,052	584.21	303,052	
Waubonsie Creek Dam	429,799	580.16	429,799	
Rakes Creek Dam	30,909	577.29	30,909	
Weeping Water Creek Dam	94,837	568.67	94,837	
Nebraska City Dam	50,699	562.74	50,699	
Horse Creek Dam	17,231	561.13	17,231	
Camp Creek Dam	66,847	549.02	66,847	

Table 2.3-5: 500-Year Flow on Missouri River at Drainage Locations

Name	Cumulative 500-Year Event Flow (cfs) On Missouri River	Contributing 500-Year Flow (cfs) on Missouri at Drainage Location
Gavins Point Dam	123,500	123,500
James River	141,500	18,000
Vermillion River	145,100	3,600
Elk Creek [1]	145,000	-100
Big Sioux River	185,400	40,300
Perry Creek	185,600	200
Floyd River	192,900	7,300
Omaha Creek	195,000	2,100
Blackbird Creek	197,300	2,300
Decatur, NE	197,700	400
Big Whisky & M&H Ditch	205,400	7,700
Little Sioux River	232,200	26,800
Tekamah Div. Ditch	233,300	1,100
Soldier River	236,500	3,200
Old Soldier R. Ditch	237,800	1,300
Fish Creek	238,800	1,000
Boyer River	245,900	7,100
Pigeon Creek	247,700	1,800
Omaha, NE	247,900	200
Mosquito Creek	248,400	500
Papillion Creek	249,000	600
Platte River	344,400	95,400
Watkins Ditch	344,700	300
Weeping Water Creek	345,300	600
Nebraska City, NE	345,400	100
Nishnabotna River	361,000	15,600
Little Nemaha River	365,900	4,900

Note:

1. Elk Creek shows a negative 100 cfs as contributing 500-year flow to Missouri River. In this analysis, as a conservative approach, a 0 cfs flow was assumed as contributing 500-year flow from Elk Creek into Missouri River.

Table 2.3-6: Peak Flow at HEC-RAS Inflow Locations

Inflow Name	Lateral Inflow 1 River Mile	Dam Failure Hydrograph Peak + 500 Yr Flow (cfs) (Non-System Dam Failure Analysis)	Dam Failure Hydrograph Peak + Mean Monthly Flow (cfs) (System and Non-System Dam Failure Analysis)
Beaver Creek	806.23	21,789	(b)(3):16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F)
Antelope Creek	803.85	73,644	
James River	797.73	138,810	
Bow Creek	787.75	2,816	
Vermillion River	771.90	24,309	
Aowa Creek	745.19	131,832	
Elk Creek	737.40	4,581	
Big Sioux River	734.00	122,241	
Perry Creek	731.76	7,644	
Floyd River	730.95	23,860	
Bacon Creek	730.54	82,505	
Pigeon Creek Ditch	720.03	142,379	
Omaha Creek	719.62	11,682	
Blackbird Creek	697.41	8,018	
Decatur Junction (1)	691.35	400	
Elm Creek	690.95	110,983	
Big Whisky Creek	669.83	408,697	
Little Sioux River	669.23	85,229	
Tekamah Creek	664.56	298,924	
Soldier River	664.00	7,505	
Cameron Ditch	658.91	29,244	
Old Soldier R. Ditch (1)	649.19	1300	
Fish Creek (1)	647.57	1000	
Long Creek	645.15	51,803	

Table 2.3-6: Peak Flow at HEC-RAS Inflow Locations, continued

Inflow Name	Lateral Inflow 1 River Mile	Dam Failure Hydrograph Peak + 500 Yr Flow (cfs) (Non-System Dam Failure Analysis)	Dam Failure Hydrograph Peak + Mean Monthly Flow (cfs) (System and Non-System Dam Failure Analysis)
De Soto Bend	641.91	28,722	(b)(3) 16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F)
Moores Creek	640.73	24,614	
Boyer River	635.22	70,710	
Deer Creek	632.99	38,270	
Pigeon Creek	622.16	32,420	
Omaha Junction [1]	616.07	200	
Mosquito Creek	605.87	82,304	
Papillion Creek	596.47	322,133	
Platte River	594.82	1,697,327	
Plattsmouth	594.14	24,120	
Watkins Ditch	587.44	300	
Pony Keg Creek	587.06	93,408	
Rock Creek	584.21	303,052	
Waubonsie Creek	580.16	429,799	
Rakes Creek	577.29	30,909	
Weeping Water Creek	568.67	95,437	
Nebraska City	562.74	50,799	
Horse Creek	561.13	17,231	
Camp Creek	549.02	66,847	
Nishnabotna River	542.10	15,600	
Little Nemaha River	527.80	4,900	

Note:

1. No outflow hydrograph from HEC-HMS model. USACE UMRSFFS report, Appendix F (Reference 2.3-11), reports 500-year flow at these locations. Thus, these inflows are present only in the Non-System dam failure analysis.

Table 2.3-7: Non-System Dam Failure Peak Stage at CNS and Plant Flood Protection Level

Plant Flood Protection Level (ft NAVD88)	Estimated Peak Stage (ft NAVD88)	Date-Time of Estimated Peak Stage	Elapsed Time to Estimated Peak Stage	Estimated Peak Flow (cfs)
906.37	903.09	1/5/3000 5:52	4 days, 5 hours, 52 minutes	1,332,697

Table 2.3-8: Peak Stage/Flow at CNS for Combined System and Non-System Dam Failure

System Dam Failure Scenario	Estimated Peak Stage (ft NAVD88)	Date and Time of Estimated Peak Stage	Elapsed Time to Estimated Peak Stage	Estimated Elapsed Time to Plant Flood Protection Level	Estimated Duration of Stage Above Plant Flood Protection Level	Estimated Peak Flow (cfs)
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Hydrologic	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)					
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Sunny Day						
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Hydrologic						
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Hydrologic						
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Hydrologic						
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Sunny Day						
(b)(3):16 U.S.C. § 824o-1(d); (b)(4), (b)(7)(F) [redacted] Hydrologic						

Table 2.3-9: Maximum Average Velocities at CNS - Combined System and Non-System Dam Failure

System Dam Failure Scenario	Maximum Average LOB [1] Velocity (fps)	Elapsed Time to Maximum Average LOB Velocity	Maximum Average Channel Velocity (fps)	Elapsed Time to Maximum Average Channel Velocity	Maximum Average ROB [2] Velocity (fps)	Elapsed Time to Maximum Average ROB Velocity
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)					
Sunny Day						
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic						
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic						
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic						
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Sunny Day						
(b)(3):16 U.S.C § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic						

Notes:

1. LOB = Left Overbank.
2. ROB = Right Overbank

Table 2.3-10: Refined Maximum Velocity at CNS - (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic and Non-System Dam Failure

River Mile	Maximum Average LOB Velocity (fps)	Elapsed Time to Maximum Average LOB Velocity	Maximum Average Channel Velocity (fps)	Elapsed Time to Maximum Average Channel Velocity	Maximum Average ROB Velocity (fps)	Elapsed Time to Maximum Average ROB Velocity
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)					

Table 2.3-11: Calculation of Wind-Driven Waves and Wind Setup Approaching CNS

Direction	Controlling Fetch Angle (deg)	Average Depth Along Fetch (ft)	Effective Fetch Length (ft)	Design Wind Speed (mph)	Wind Speed Duration (min)	H_{mo} (ft)	T_p (s)	Wavelength (ft)	Wind Setup (ft)	$H_{1\%}$ (ft)
Plant North	0.0	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	75,564.30	44.6	55	6.10	4.71	112.37	0.44	10.19
Plant East	135.0		137,392.09	43.6	85	7.66	5.37	144.97	0.66	12.79
Plant South	135.0		137,392.09	43.6	85	7.66	5.37	144.97	0.66	12.79
Plant West	157.5		135,273.58	43.6	85	7.04	5.16	133.71	0.71	11.76

Table 2.3-12: Summary of Wind Setup, Wave Runup, and Total Water Levels at SSCs

SSC	Wind Direction	Still Water Level (ft NAVD88)	Wind Setup (ft)	Wave Runup (ft)	Maximum Water Level, including Setup and Runup (ft NAVD88)
Intake Structure	Plant East	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	0.66	19.19	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
Control Building	Plant South		0.66	19.19	
Reactor Building	Plant South		0.66	19.19	
Radwaste Building	Plant West		0.71	17.64	
Diesel Generator, Heating and Fan Building	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)				

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Table 2.3-13: Summary of Water Depth at Important Plant Structures

Structure Name	Roof Elevation (ft NAVD88)	WESL (ft NAVD88)	Water Depth (ft)
Radwaste Building	942.5		
Control Building	949.5		
Reactor Building	1,048.3		
Office Building	933.2		
Turbine Generator Building	1,005.9		
Intake Structure	941.5		
Maintenance Shop	948.3		
Water Treatment Building	948.3		
Diesel Generator, Heating and Fan Building	931.9		
Controlled Corridor	949.5		

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Table 2.3-14: Summary of Water Depth at Important Bridges

Bridge Name	Estimated Bridge Deck Elevation (ft NAVD88)	WESL (ft NAVD88)	Water Depth	Duration of Inundation
NE State Hwy 67 and Little Nemaha River (near Brock, NE)	959			
US Hwy 75 and Little Nemaha River	935			
US Hwy 136 and Little Nemaha River	927			
US Hwy 75 and South Table Creek	975			
NE State Hwy 67 and Little Nemaha River (near Nemaha, NE)	903			
State HWY 67 and Honey Creek	951			

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Table 2.3-15: Resultant Forces and Elevations on Main Building Complex Structures

	Hydrostatic		Hydrodynamic Crest		Hydrodynamic Trough		Current-Induced	
Direction	**Force (lb/ft of wall)**	**Resultant Force Elevation (ft NAVD88)**	**Force (lb/ft of wall)**	**Resultant Force Elevation (ft NAVD88)**	**Force (lb/ft of wall)**	**Resultant Force Elevation (ft NAVD88)**	**Force (lb/ft of wall)**	**Resultant Force Elevation (ft NAVD88)**
Plant North	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)							
Plant East								
Plant South								
Plant West								

Table 2.3-16: Resultant Forces and Elevations on Intake Structure

Structure Type	Force (lb/linear ft of wall)	Resultant Force Elevation (ft NAVD88)
Hydrostatic	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	
Hydrodynamic crest		
Hydrodynamic trough		
Current-induced		

Table 2.3-17: Debris Impact Loads

Maximum Velocity	Debris Source	Impact Load (kip)
Channel (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) fps)	1000# miscellaneous debris	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
	4000# large natural debris	
	Large vehicles and boats	
	Tank-type debris	
	Barge	
Overland fps)	1000# miscellaneous debris	
	4000# large natural debris	
	Large vehicles and boats	
	Tank-type debris	
	Barge	

2.3.8 Figures

Figures associated with Section 2.3 are presented on the following pages.

Figure 2.3-1: Non-System Dams Downstream of Gavins Point Dam

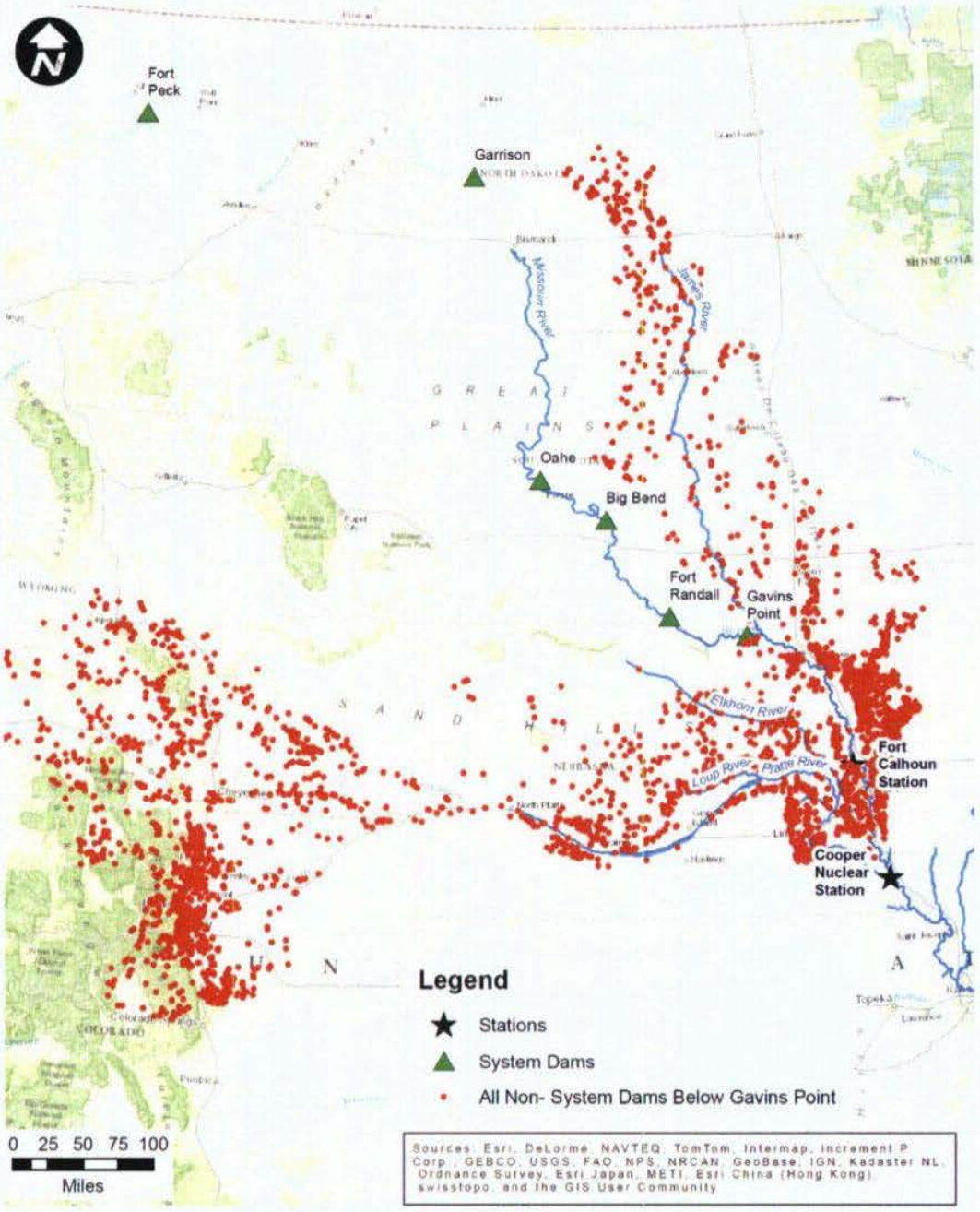


Figure 2.3-2: Inconsequential Non-System Dams Downstream of Gavins Point Dam

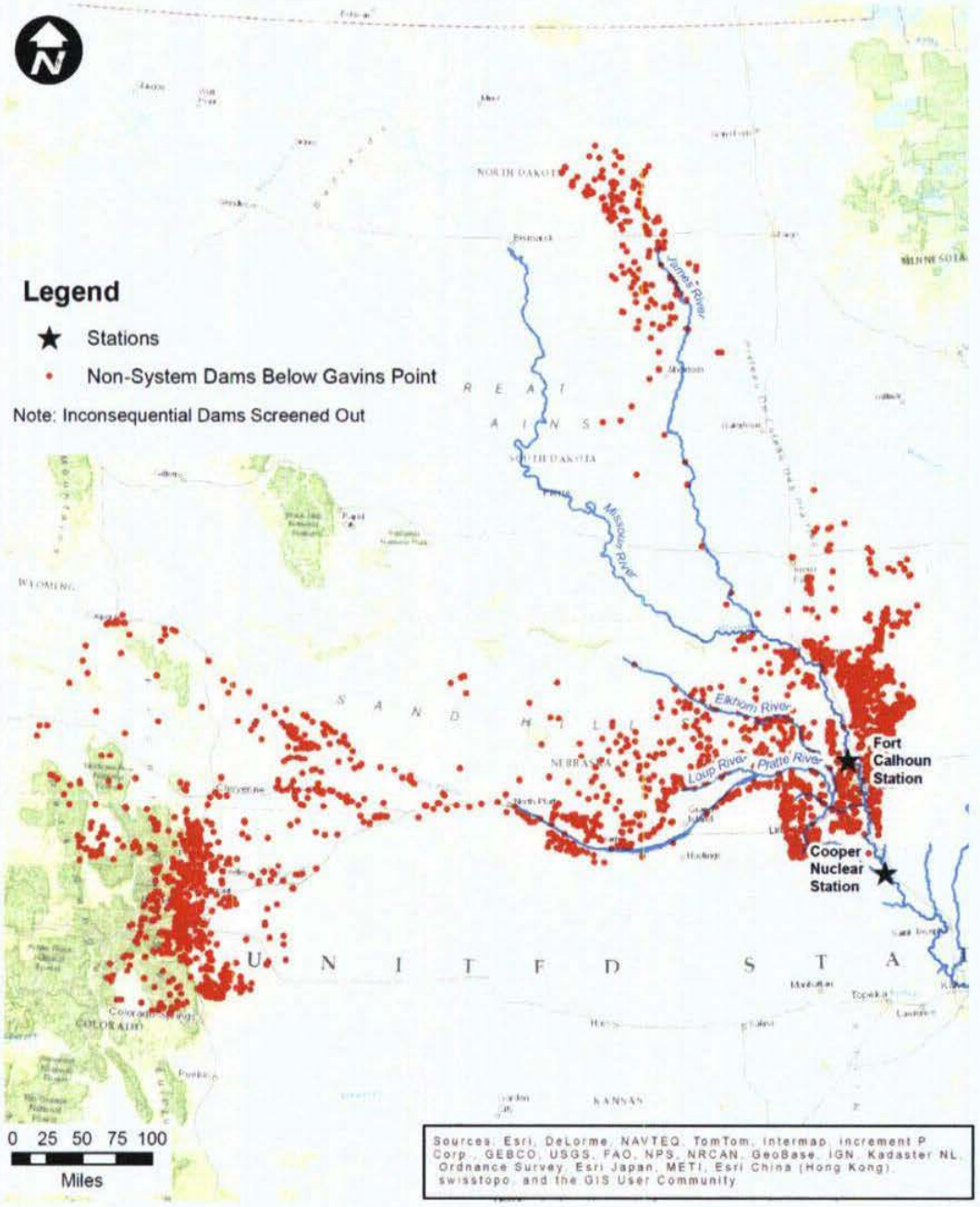


Figure 2.3-3: HEC-HMS Plan for Missouri River below Gavins Point Dam to Omaha, NE

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-4: HEC-HMS Plan for Missouri River from Omaha, NE to CNS

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-5: Dam Failure Hydrographs at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **Dam due to System Hydrologic Dam Failure**

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-6: Dam Failure Hydrographs at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **Dam due to System Sunny-Day Dam Failure**

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-7: Selected Dam Failure Hydrographs at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **Dam due to System Dam Failure**

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-8: Estimated Peak Stage and Peak Flow at CNS for Non-System Dam Failure

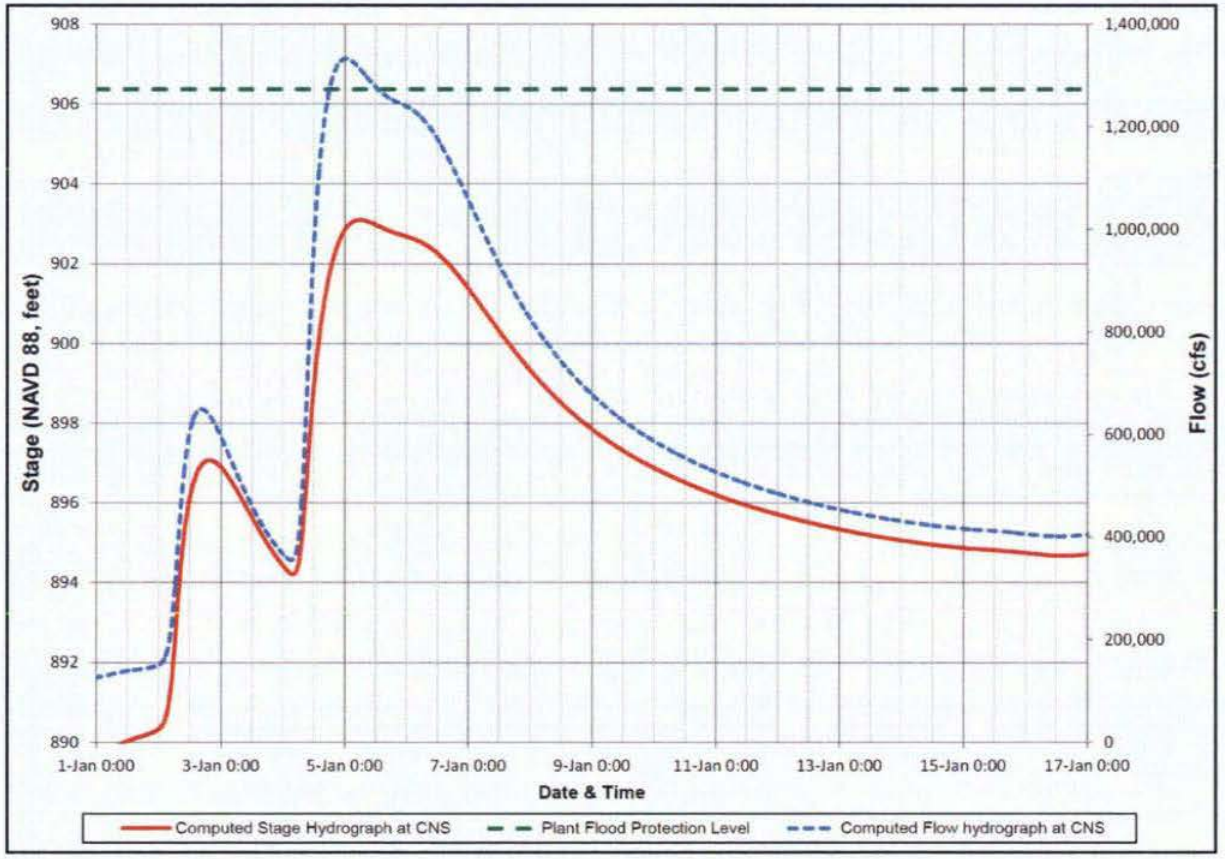


Figure 2.3-9: Estimated Peak Stage at CNS due to System and Non-System Dam Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-10: Estimated Peak Discharge at CNS due to System and Non-System Dam Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-11: Velocity Distribution Segments at HEC-RAS Cross Section 532.65 (CNS Site)

(b)(3):16 U.S.C. § 824o-1(d); (b)(4); (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-12: Max [redacted] Bank Velocity & Total Discharge - [redacted] Hydrologic Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-13: Max. Channel Velocity & Total Discharge - (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Hydrologic Failure (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-14: Max. Bank Velocity & Total Discharge Hydrologic Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-15: WSEL with Fetch Overlay

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-16: Fetch Directions for Waves Approaching Plant East and Plant West

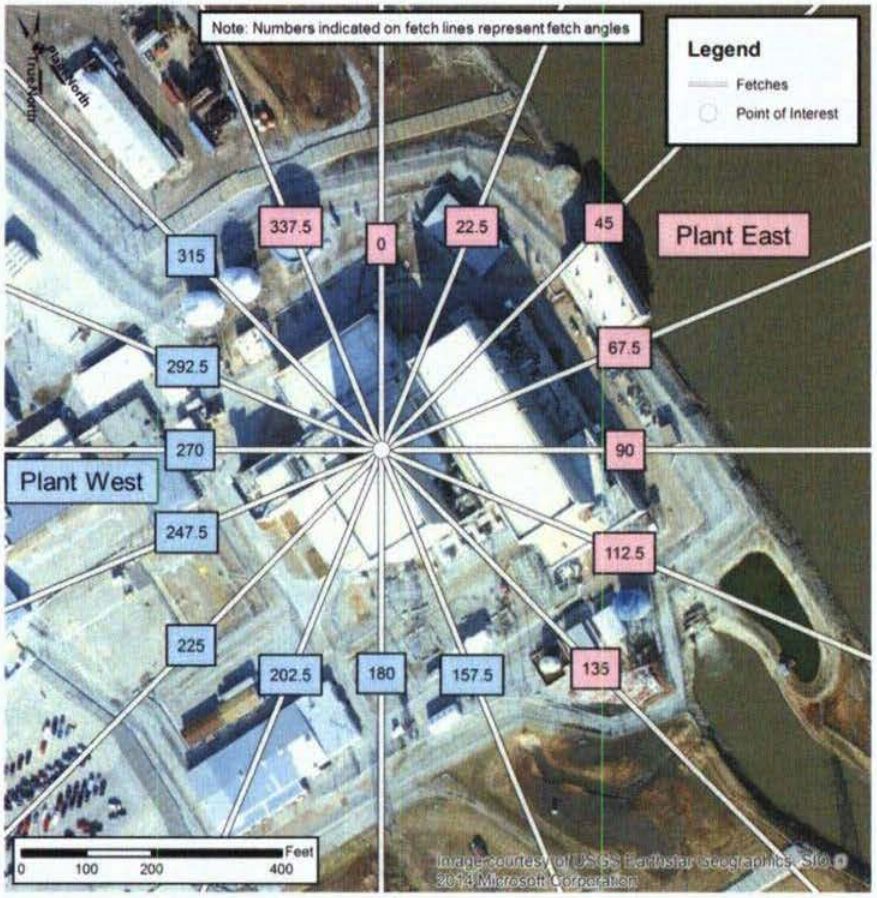


Figure 2.3-17: Fetch Directions for Waves Approaching Plant North and Plant South

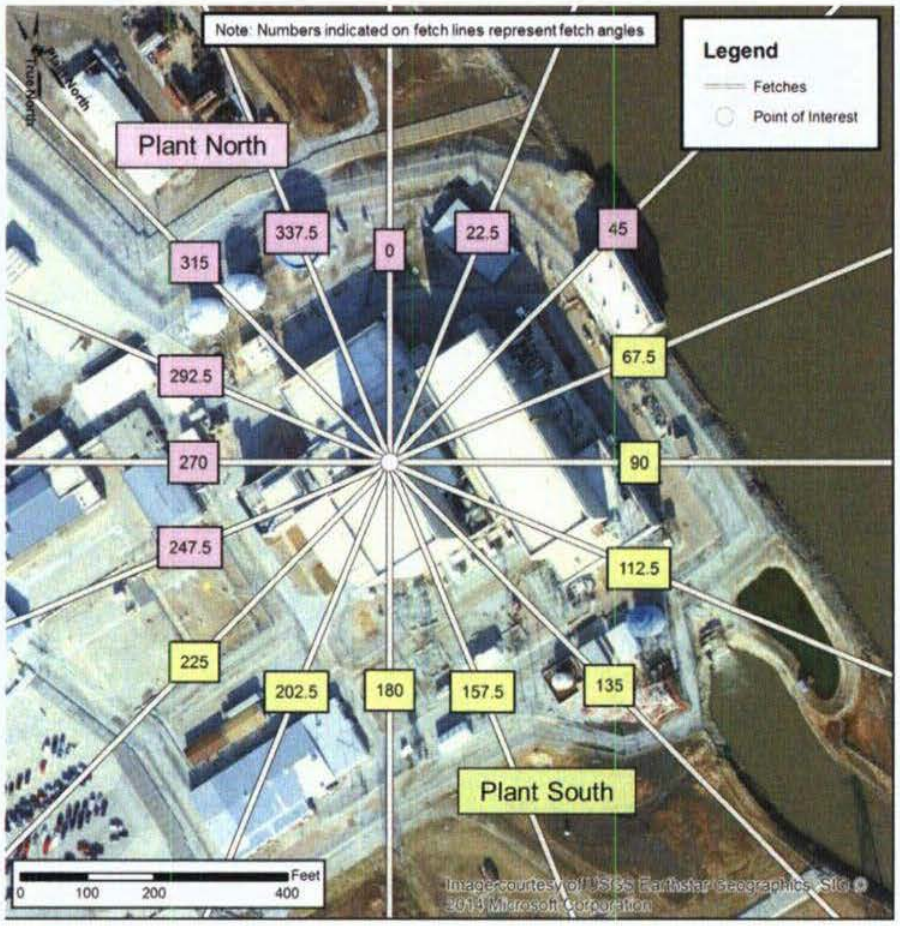


Figure 2.3-18: Plan View of Pressure Calculation Surfaces for (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-19: Cross Sections and Channel Bank Locations at CNS RMs 532.65, 532.53, and 532.49

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-20: Max. Bank Velocity & Total Discharge - Hydrologic Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-21: Water Surface Elevation & Total Discharge (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **Hydrologic Failure** (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-22: Max. Bank Bed Shear & Total Discharge Hydrologic Failure

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-23: Inundation Map for Hydrologic Dam Failure Scenario (Large-Scale Map)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-24: Inundation Map for Hydrologic Dam Failure Scenario (Medium-Scale Map)

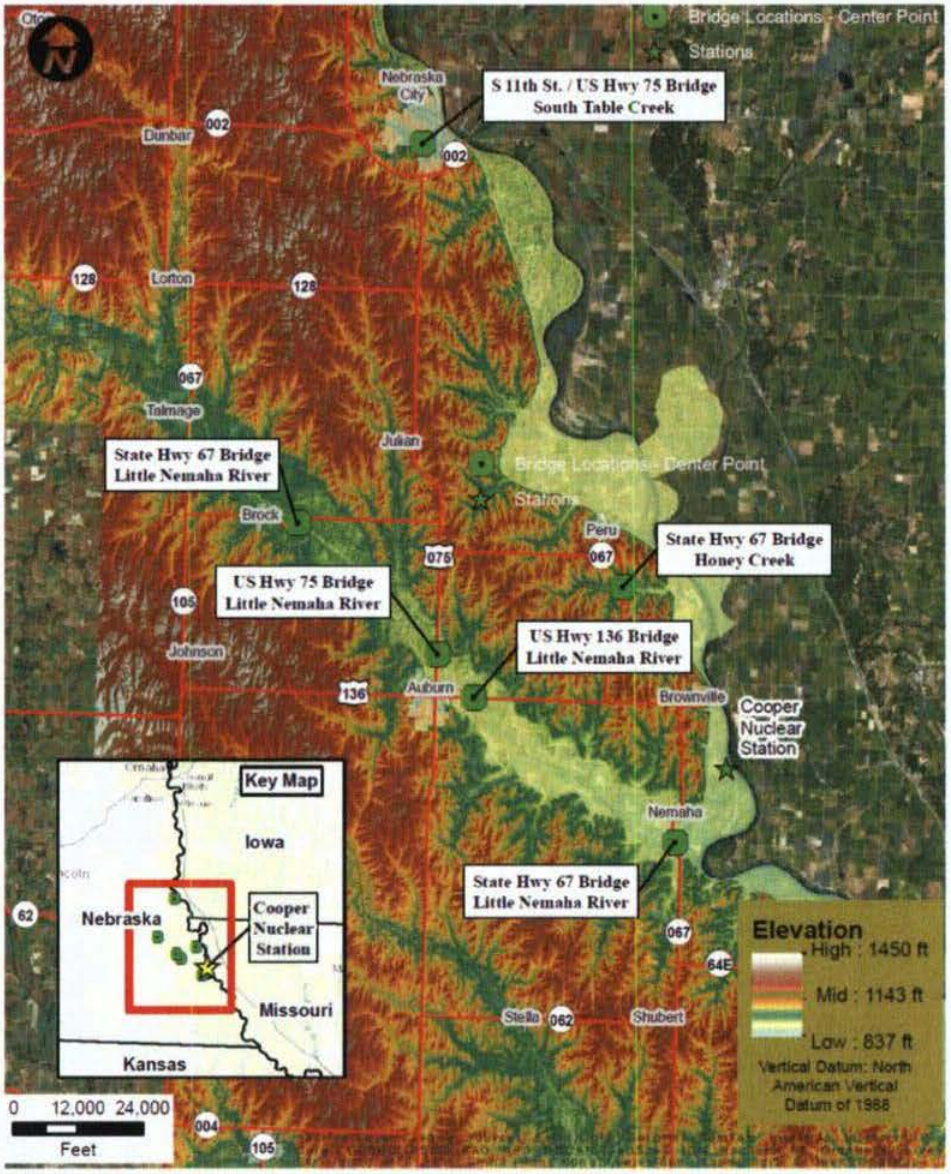
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-25: Inundation Map for Hydrologic Dam Failure Scenario (Small-Scale Map)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Figure 2.3-26: Bridge Location Map



2.4 STORM SURGE

Storm surge is a rise in offshore water elevations caused principally by the shear force of winds acting on the water surfaces, typically associated with tropical storms (Reference 2.4-1, Section 3.5). The Cooper Nuclear Station (CNS) is not near any large bodies of water for which storm surge flooding would apply; therefore, the risk to the plant from a storm surge event that could cause flooding at CNS is not expected to be a potential flooding hazard. According to ANSI/ANS-2.8-1992 guidance (Reference 2.4-2 2), the region of occurrence of a hurricane shall be considered for United States coastline areas and areas within 100 to 200 miles bordering the Gulf of Mexico. Because CNS is located far from any coastline and at a site grade of 903.37 ft NAVD88, hurricanes do not present potential flooding hazards.

Therefore, storm surge was screened out as a credible flooding mechanism for CNS.

2.4.1 References

2.4-1. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

2.4-2. American Nuclear Society. 1992. ANSI/ANS-2.8-1992: Determining Design Basis Flooding at Power Reactor Sites. American Nuclear Society Publishing, La Grange Park, IL.

2.5 SEICHE

A seiche is an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause. Once started, the oscillation may continue for several cycles; however, over time it gradually decays because of friction (Reference 2.5-1, Section 3.6).

The potential flooding hazard from a seiche at the Cooper Nuclear Station (CNS) is judged to be negligible because of the site's riverine setting. The Missouri River channel in the CNS area is narrow (less than 0.5 mile), shallow (35 ft or less), and meandering, which constrains and limits the geometry needed to develop a seiche and its oscillation propagation. The river geometry also limits the height of any seiche oscillations and causes rapid attenuation of any seiche oscillations.

Therefore, seiche was screened out as a credible flooding mechanism for CNS.

2.5.1 References

2.5-1. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

2.6 TSUNAMI

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources (Reference 2.6-1, Section 1.1). As an inland site, the Cooper Nuclear Station (CNS) is not susceptible to oceanic tsunamis (Reference 2.6-1, Section 2.1).

Therefore, tsunami was screened out as a credible flooding mechanism for CNS.

2.6.1 References

2.6-1. Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America, NUREG/CR-6966, Office of New Reactors, U.S. Nuclear Regulatory Commission, Washington, D.C.

2.7 ICE-INDUCED FLOODING

As per the U.S. Nuclear Regulatory Commission (NRC) guideline (NUREG/CR-7046, Reference 2.7-1), ice jams and ice dams can form in rivers and streams adjacent to a site and may lead to flooding by two mechanisms: (1) collapse of an ice jam or an ice dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site and (2) an ice jam or an ice dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

2.7.1 Methodology

The NUREG guidance (Reference 2.7-1) identifies that, while it is possible to assess whether a site may possess hydroclimatic conditions that are precursors to ice jam or ice dam formation, it is not possible to accurately predict the exact location and severity of the ice blockage. Therefore, it is not possible to accurately predict a probable maximum ice jam or ice dam. Alternatively, it is recommended that historical records of ice jams and ice dams be searched to determine the most severe historical event in the vicinity of the site.

The hierarchical hazard assessment (HHA) approach described in Reference 2.7-1 was used for the evaluation of the effects of ice-induced flooding on the water surface elevation (WSEL) at Cooper Nuclear Station (CNS). In regard to ice-induced flooding on the Missouri River, the HHA used the following steps:

1. Identify the largest historic ice-induced flooding event and calculate ice jam height upstream and downstream of the site.
2. Conservatively calculate peak water surface elevation resulting from failure of an upstream ice jam.
3. Conservatively calculate peak water surface elevation from backwater effects resulting from a downstream ice jam.

2.7.2 Most Severe Historical Ice Jam Event

The U.S. Army Corps of Engineers (USACE) National Ice Jam Database (Reference 2.7-2) was searched to determine the most severe historical ice jam event in the Missouri River and its major tributaries downstream of Gavins Point Dam, such as the Missouri, James, Big Sioux, Little Sioux, Elkhorn, Platte, and Nishnabotna rivers (Figure 2.7-1). U.S. Geological Survey (USGS) hydrologic unit code (HUC) boundaries were further utilized to identify the regional search area. The ice jam record represents a USGS gage location (Reference 2.7-3), which provides a gage height for the ice jam event. Using the datum of the gage and ice jam gage height, the peak flood elevation was determined. A normal WSEL was conservatively estimated for each USGS gage using the minimum value from the field measurement data. The difference between the peak elevation and normal WSEL was used as an initial estimate of the ice jam height. If the initial estimate of the ice jam's height presents a challenge, the normal WSEL is refined more rigorously based on a mean monthly gage height during the cool-season. The largest historic ice-induced upstream flooding event occurred 200 miles upstream of CNS on April 23, 1881 at Sioux City, Iowa. The estimated ice dam height was 21.75 ft at RM 732.2, which is

equivalent to a stage of 1079.48 ft National Geodetic Vertical Datum of 1929 (ft NGVD29).The largest historic ice-induced downstream flooding event occurred 34.5 miles downstream of CNS on December 29, 1972 at Rulo, Nebraska. The estimated ice jam height was 14.45 ft at RM 498, which is equivalent to a stage of 857.97 ft NGVD29.

2.7.3 Upstream Breach of an Ice Dam

For an ice jam collapse, the National Weather Service (NWS) simplified dam failure equation (Reference 2.7-4) was used to determine a peak outflow. The outflow was transposed to CNS without attenuation. Although NUREG/CR-7046 (Reference 2.7-1) does not specify any coincident flows during an ice jam breach, the cold season (December through March) mean monthly flow was added to this flow to calculate a conservative discharge. A corresponding WSEL was then determined based on the stage-flow frequency data for the Missouri River at different River Miles (RMs) from the Upper Mississippi River System Flow Frequency Study ([UMRSFFS] Reference 2.7-5). The peak outflow was then calculated, after evaluating all seven rivers, for an ice dam forming and breaching at the US Highway 77 and Missouri River Bridge (called Sioux City Highway Bridge) located at RM 732.38.The resultant calculated peak flow (breach plus mean monthly) at the Sioux City Highway Bridge, as a result of the historic 21.75-foot high ice jam, was estimated as159,075 cfs. The peak WSEL at CNS due to an ice jam forming and breaching at the Sioux City Highway Bridge is 896.54 ft NGVD29 (896.8 ft NAVD88).

2.7.4 Downstream Ice Jam and Resulting Backwater

For ice jam backwater, the stage-flow frequency (Reference 2.7-5) data were first used to determine the associated flood flow at the gage downstream of the site based on the ice jam height. Then the cold season (December through March) mean monthly flow at CNS was added to establish a conservative flood flow. The associated stage for such flow is found in Reference 2.7-5 at the gage. In order to postulate the established flood stage at CNS, first the maximum water surface slope was calculated between the two locations (based on Reference 2.7-5), one located at the ice jam and the other at the site. That maximum slope was then used to translate the downstream flood stage to the WSEL at CNS. The maximum WSEL at CNS due to backwater from the most severe ice jam forming in the Missouri River near Rulo, Nebraska was 896.6 ft NGVD29 (896.86 ft NAVD88).

2.7.5 Effect of Ice-Induced Flooding

The peak WSEL at CNS resulting from the upstream ice jam / ice dam breach was conservatively calculated to be 896.54 ft NGVD29 (896.8 ft NAVD88). The peak WSEL at CNS as a result of backwater caused by the ice jam at Rulo, Nebraska was conservatively calculated to be 896.6 ft NGVD29 (896.86 ft NAVD88). According to Reference 2.7-6, the finished floor elevations for Principal Class 1 Structures are equal to 903.5 ft Plant Datum, which is equal to 903.66 ft NGVD29 (903.92 ft NAVD88). Therefore, the maximum ice-induced flood water level will still be 7.06 ft below the minimum floor elevation for all of the safety related facilities.

2.7.6 References

2.7-1. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, PNNL-20091, United States Nuclear Regulatory Commission (USNRC), November 2011.

2.7-2. Cold Region Research Engineering Laboratory (CRREL), U.S. Army Corps of Engineers (USACE), National Ice Jam Database, Bulletin and Survey, Website http://icejams.crrel.usace.army.mil/ accessed June 2014.

2.7-3. United States Geological Survey (USGS), National Water Information System (NWIS), Web Interface, USGS Water Data for the Nation, Website http://waterdata.usgs.gov/nwis, accessed June 2014.

2.7-4. The NWS Simplified Dam-Break Flood Forecasting Model, Abstract, by Jonathan N. Wetmore and Danny L. Fread, Office of Hydrology, National Weather Services, NOAA, Maryland. http://www.nws.noaa.gov/oh/hrl/hsmb/docs/hydraulics/papers_before_2009/hl_154.pdf

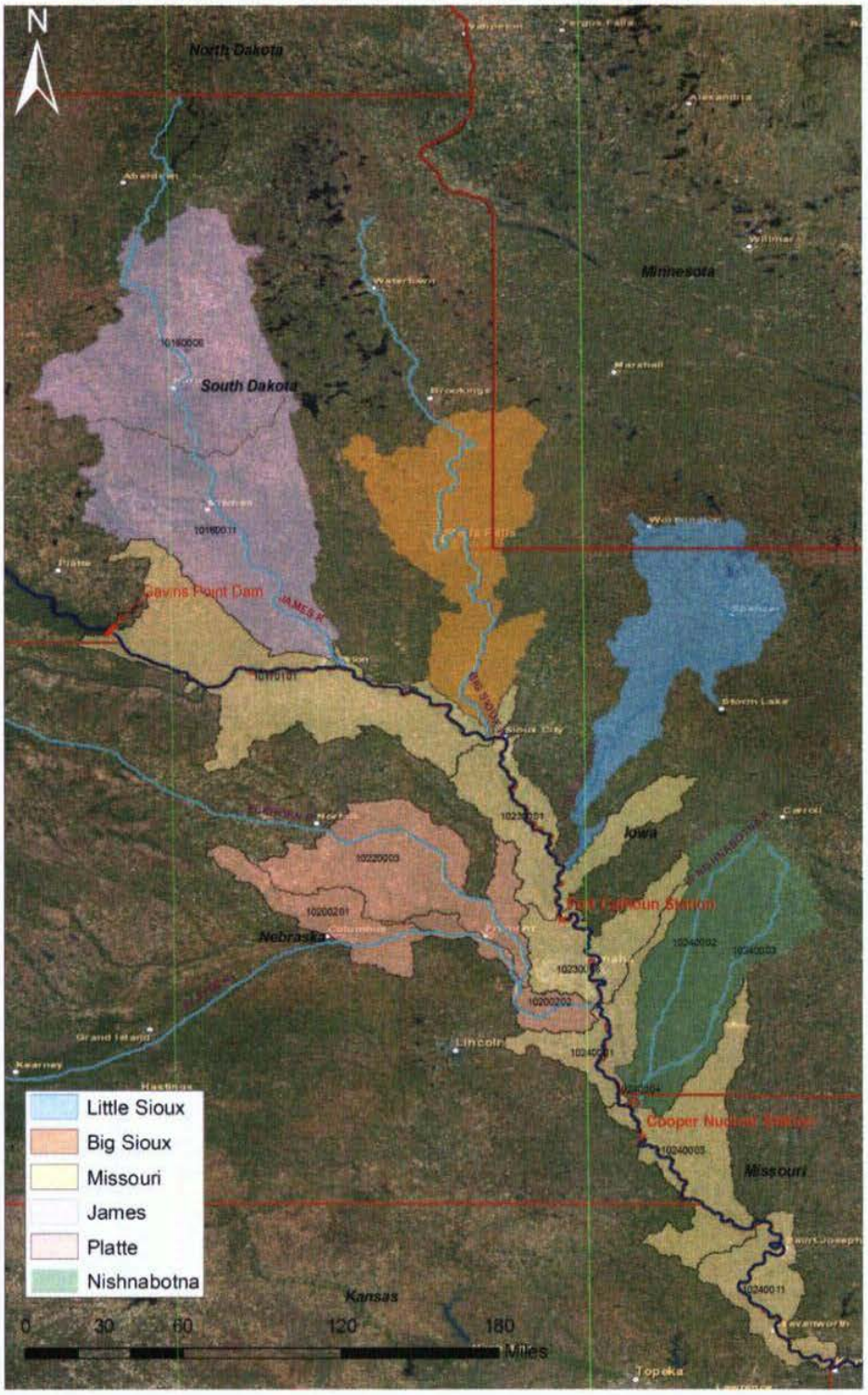
2.7-5. Upper Mississippi River System Flow Frequency Study, Hydrology and Hydraulics Appendix F, Missouri River, U.S. Army Corps of Engineers, Omaha District, November 2003.

2.7-6. Cooper Nuclear Station, Updated Safety Analysis Report (USAR), Section II-4, p II-4-5 (01/16/01); and Section XII-2, p XII-2-1 (02/18/05) and p XII-2-6 (02/18/05).

2.7.7 Figure

The figure associated with Section 2.7 is presented on the following page.

Figure 2.7-1: Hydrologic Unit Code (HUC) for Different Watersheds Upstream and Downstream of the Site



2.8 CHANNEL MIGRATION OR DIVERSION

Natural channels may migrate or divert either away from or toward the site, and the relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, as suggested in NUREG/CR-7046 (Reference 2.8-1) historical records and hydrogeomorphological data are used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site.

2.8.1 Historical Channel Migration or Diversion

Prior to regulation, the Missouri River, a meandering alluvial river, carried large amounts of sediment. Streambank erosion tended to be most severe as flood waters were rising, with substantial deposition of sediment occurring as flood waters receded. The Missouri River existed in a dynamic equilibrium within its floodplain, frequently redistributing sediment between its channel and floodplain (Reference 2.8-2).

As Missouri River flows increased in the spring and summer, the river would erode sediment from its bed and its banks. The river bed would undergo rapid physical changes during this period; degrading due to erosion, the channel would migrate laterally, backwaters and the main river channel would be connected by overbank flows, and banks would erode and wash downstream. As flows receded (the hydrograph's receding limb) and water volume and velocity decreased, the degraded channel would refill with sediment, the braided channels and meanders would become isolated from the main channel, and fresh substrates would be deposited (Reference 2.8-2).

A typical cross section of the Missouri River prior to construction of the System dams contained a deep channel, multiple side channels, oxbow lakes, islands, sandbars and dunes, and backwater habitats interspersed by areas of higher land (Reference 2.8-3). Figure 2.8-1 shows the Missouri River in 1879 in the area where Cooper Nuclear Station (CNS) was constructed (Reference 2.8-3). The river meandered eastward just east of the future CNS site and then flowed in a southwesterly direction for several miles downstream.

The variability that characterized hydrology of the Missouri River prior to construction of the System dams had diminished (as it is shown in Figure 2.8-1 to Figure 2.8-8) along most of the river following the Flood Control Act of 1935 and the Rivers and Harbors Act of 1945. U.S. Army Corps of Engineers (USACE) had channelized most of the Missouri River through a combination of engineering structures, natural and engineered hard points, revetments, and dikes. Construction of the lower five Missouri River mainstem dams began in the 1950s, and the reservoirs were filled to operating levels by 1967 (Reference 2.8-4). Figure 2.8-3 shows an aerial image dating from 1965 (Reference 2.8-5), showing the Missouri River navigational channel being established, and bank stabilization structures visible on both banks of the river. Most of the old meanders and side channels on the west side of the river were no longer visible. Additionally, levees had been constructed on both the eastern and western banks of the river.

By the 1980s and 1990s, the navigational channel had been established for decades, and the banks no longer migrated and were highly vegetated. On July 23, 1993, the U.S. Geological Survey (USGS) measured a flow of 196,000 cubic feet per second (cfs) at the gage at Nebraska City, Nebraska (Reference 2.8-6). This is the second highest flow measurement since the construction of the System dams. Neither the 1993 image (Figure 2.8-4) nor the 1999 image (Figure 2.8-5) show any evidence of floodplain inundation or channel diversion from this event. Aerial photography of the CNS site and the Missouri River upstream and downstream for the years 1993, 1999, 2003, and 2005 are shown in Figure 2.8-4 through Figure 2.8-7, respectively. These images show a stable, unmoving channel. The banks, having not migrated for over four decades, were highly vegetated. Bank stabilization was firmly in place and being maintained. There were no visible changes to the banks or old meanders to the north or south of the CNS site during this time.

An aerial image dating from 2010 (Figure 2.8-8, Reference 2.8-7) shows inundation of the floodplain in the old channel meanders to the north and to the south of the CNS site. The image also shows inundation of the floodplain on the east side of the river. The peak flow measured at the USGS gage at Nebraska City, Nebraska, in 2010 was 168,000 cfs (Reference 2.8-6) and is one of the top ten largest recorded flows since the construction of the System dams. There is no evidence in this image of the main navigation channel migrating in any way. The water that was inundating the floodplain showed no evidence of forming new channels or meanders. On the west bank, there was no visible river inundation west of the levee. On the east bank, there was ponding water east of the levee. This was likely due to interior drainage ponding.

2.8.2 Regional Topographic Evidence

The following information was used in conjunction with the historical channel diversion information documented in Section 2.8.1 to evaluate whether a future diversion may or may not occur. The results were used to assess the possibility of channel diversion near the site that may affect CNS.

2.8.2.1 Literature Review

NUREG-0800 (Reference 2.8-8) references two documents that suggest methodologies for assessing channel diversion potential. The first document is titled "Engineering and Design Channel Stability Assessment for Flood Control Projects" and is published by USACE (Reference 2.8-9). The second document is titled "Methodology for Predicting Channel Migration" and is published by National Cooperative Highway Research Program (NCHRP) (Reference 2.8-10). These documents were reviewed, and pertinent information was used for the qualitative geomorphic analysis. The techniques discussed in these two documents are most useful for natural channels. USACE constructed river training structures and bank stabilization, and continues to maintain a self-sustaining navigational channel. Therefore, future meandering is not anticipated outside of what might occur during flood events (Reference 2.8-9).

USACE's Missouri River Stage Trends Technical Report (Reference 2.8-11) was also reviewed to establish the current or near-future state of the river near CNS as aggradational, degradational, or stable. The stage trend, or specific gage analysis, is performed to evaluate trends in bed elevation based on a recorded stage for a given discharge over time. The measurements (Reference 2.8-6)

between 0 and 100,000 cfs show a stable channel, and the measurements above 100,000 cfs recorded in 2012 and later also conform to the same general trend of the measured data of the previous rating curve and, therefore, indicate channel stability. The 2011 measurements with discharges above 150,000 cfs seem to show channel degradation.

2.8.2.2 Soil Survey Data

Figure 2.8-9 through Figure 2.8-11 (References 2.8-12 and 2.8-13) show hydric soil data, geomorphic classification of the soil data, and soils texture in the vicinity of the CNS, respectively.

Hydric soils are highlighted and clearly show locations where the old Missouri River meandered and the old channels had been. Traces of the old meanders on the west side of the Missouri River, both to the north and south, can be seen in the hydric soil data. On the east bank, hydric soils are contained within the boundary of the levee, although the Missouri River had been east of the levee.

The current channel is classified as water, and the old meanders and the entire area between the levees are classified as floodplains. On the Nebraska side, west of the levee, the area is classified as uplands, while on the Missouri side, east of the levee, the area is classified as river valley and terraces.

The textures include combinations of sand, clay, silt, and loam. These soil textures were used to compare allowable velocities.

The USDA NRCS gSSURGO database (References 2.8-12 and 2.8-13) contains a field for soil erosivity; however, the values for the soils in the vicinity of CNS are listed as either null or none. Therefore, soil erosivity in the vicinity of CNS was not evaluated.

2.8.2.3 Missouri River Bank Stabilization and Navigation Maps

The Missouri River navigational charts (Reference 2.8-14) show that bank stabilization structures and levees have been placed all along the Missouri River channel. In the immediate vicinity of CNS, stabilization of the bank along the concave alignment of the design curve was accomplished with pile and stone fill revetments. Dikes were constructed along the convex bank, approximately perpendicular to the flow. These dikes were designed to prevent bank erosion and to promote accretion, forcing the channel to develop and maintain itself along the design alignment.

2.8.2.4 2011 Flood Documentation

The 2011 flood is the flood of record since the System dams were constructed. USACE developed the After Action Report, which chronicled emergency actions taken during the 2011 flood (Reference 2.8-15).This document describes several incidents regarding the overtopping and breaching of levees upstream of CNS.

In addition, flooding at CNS as a result of potential diversion was assessed by USACE, as recommended by NUREG-0800 (Reference 2.8-8). A USGS document titled "Geomorphic Changes Caused by the 2011 Flood at Selected Sites Along the Lower Missouri River and Comparison to Historical Floods" (Reference 2.8-16) was reviewed as well.

Satellite images (Figure 2.8-12, Reference 2.8-17) show that flood waters had inundated the floodplain in many of the areas where the Missouri River had once meandered within the boundaries of the levees.

During the 2011 flood, significant overbank flooding resulted in some erosion of the eastern overbank, but the channel was contained within the levees. This is consistent with the analysis reported in Reference 2.8-16. Pronounced changes in channel bed were noted as a result of the 2011 flood at the three stream gages evaluated: Sioux City, Iowa; Omaha, Nebraska; and Kansas City, Missouri. However, channel width change was not evident at the gage locations, likely the result of the Missouri River bank stabilization and navigation projects (BSNP). In summary, emergency technical assistance was provided by USACE to CNS personnel during the 2011 flood event, and measures were implemented to allow the plant to remain fully operational.

2.8.3 Ice Causes

Cold Regions Research and Engineering Laboratory (CRREL) maintains a database of ice jam and ice buildup incidents. The database was queried for the Missouri River in the vicinity of CNS to document the most severe ice-induced conditions with respect to diversion, flooding, and low flow conditions.

According to the CRREL database (Reference 2.8-18, several ice events have occurred along the Missouri River. The CRREL information is listed at each USGS gage; however, the location of the ice condition (upstream or downstream of the gage) is not provided. Flooding from ice effects (further described in Section 2.7) near CNS, and close enough to CNS to cause increased WSELs and/or channel diversion, will not be exacerbated by releases from Gavins Point Dam (Reference 2.8-4). Therefore, flooding should be a concern only when an ice jam is combined with a rain-induced flood event in the uncontrolled tributaries downstream of Gavins Point Dam. Given this information, a channel diversion due to an ice jam is possible.

2.8.4 Flooding of Site Due to Channel Migration or Diversion

The Hydrologic Engineering Center River Analysis System (HEC-RAS) steady-state and unsteady models used in Sections 2.2 and 2.3 provide information on flow distribution and hydraulic characteristics for bankfull flow, peak flow during the PMF, and flooding due to upstream dam failures. The field measurements (collected by USGS [Reference 2.8-6]) and the HEC-RAS model results show that bankfull flow is most likely between approximately 80,000 and 100,000 cfs. The channel and overbank velocities were obtained by executing the HEC-RAS steady-state model for a discharge of 100,000 cfs.

To evaluate channel erosion potential, USACE suggests using an allowable velocity approach (Reference 2.8-9). Figure 2.8-11 shows the soil types in the vicinity of CNS. The area soils are reported as a mix of silty clays, loams, and sands, which have allowable velocities ranging from 2 to 6 fps (Reference 2.8-9). The velocities in the left and right overbanks are almost zero because at bankfull condition (100,000 cfs), there is no flow in the overbanks at this cross section. The velocities in the channel for the bankfull condition range from just under 0.4 fps to 7.7 fps. For the PMF condition (approximately 769,125 cfs), the velocities in the left overbank are less than 2.4 fps, while the velocities

in the right overbank are generally less than 0.8 fps. For the PMF condition, the velocities in the channel range between 5.4 fps and 9.6 fps. For flooding due to upstream dam failures (approximately [redacted] cfs), the velocities in the left overbank range between 2.9 fps and 5.6 fps, while the velocities in the right overbank range between 1.4 fps and 1.84 fps. The velocities in the channel range between 10.6 fps and 14.2 fps. The velocities predicted by the HEC-RAS model for the bankfull, PMF, and flooding due to upstream dam failure conditions are generally greater than the allowable velocities for the various soil textures, indicating erosion and diversion potential.

(b)(3): 16 U.S.C. § 824o-1(d), (b)

2.8.5 Human-Induced Changes of Channel Diversion

The BSNPs were authorized under various Congressional acts since 1912. Fort Peck Dam was authorized under the Rivers and Harbors Act of 1935. The lower five dams—Garrison, Oahe, Big Bend, Fort Randall, and Gavins Point—were authorized under the Flood Control Act of 1944. Additional BSNPs were authorized by the Flood Control Acts of 1941, 1946, 1948, 1963, 1968, 1974, and 1978. Further streambank erosion controls were authorized under the Water Resources Development Acts of 1974, 1986, and 1988. The navigational channel project was officially declared finished in 1981, with the terminus of the project at RM 734.8 at Sioux City, Iowa (References 2.8-4 and 2.8-19). USACE actively maintains the channel. It is unknown whether USACE plans to revise channel maintenance procedures or the System dams. The potential of channel diversion from human-induced causes is not applicable because any channel diversion for the CNS plant site would likely include coordination with U.S. Nuclear Regulatory Commission (NRC) and CNS staff prior to construction.

2.8.6 Conclusions

Prior to the construction of the Missouri River System dams and the Missouri River BSNP, regular floods occurred along the Missouri River, and the channel continually meandered as banks were eroded on one side and sediment was deposited on the other. Historical images show signs of meanders to the north and south, and the channel progressing westward in the vicinity of CNS.

Analysis of field measurements shows that the navigational channel has been through aggradational and degradational trends, and is currently generally either stable or slightly aggrading. During the 2011 flood, the channel degraded, but between 2012 (Figure 2.8-13) and 2014, has shown signs of recovery.

The soils in the channel are erodible at the velocities experienced during bankfull flow. In addition, the soils in the channel and possibly in the overbank are erodible at the velocities that may be experienced during a PMF and/or flooding due to upstream dam failures. During the 2011 flood, significant overbank flooding resulted in some erosion of the eastern overbank, but the channel was contained within the levees. Levee breaches (for location see Figure 2.8-14) that occurred upstream of CNS caused flooding behind the levees on the eastern side of the river.

This information, in conjunction with the historical meander patterns, indicates that the potential exists for the Missouri River to temporarily rechannelize into an old meander or divert away from the current navigational channel during the PMF and/or the flooding due to upstream dam failures. However, during bankfull flow, there is no significant possibility for channel migration due to the presence of the Missouri River BSNP.

Given that erosion occurred along the eastern overbank of the Missouri River and that several levee breaches were documented for peak flows of approximately 229,000 cfs, it appears possible that the channel could migrate either east or west of its current position during the PMF with maximum flow of 769,135 cfs. This would likely result in the loss of the ultimate heat sink because the CNS Intake Structure would no longer be able to access water from the Missouri River. In addition, it is possible that the PMF flows could cause the channel to migrate either east and away from its current position, or west of its current position and onto the CNS site. This could damage the functionality of other SSCs due to erosion on the CNS site that could damage underground utilities required for the SSCs to function as designed.

Because the flooding due to upstream dam failures has a potential flow of approximately 7,973,910 cfs, it is possible that the Missouri River channel would migrate away from the CNS site, through the CNS site, or both during different stages of the flood. Therefore, the potential for channel diversion during a flood event due to upstream dam failures could affect safety-related SSCs.

With the construction of the System dams and the Missouri River BSNP, the Missouri River channel has been stabilized and has maintained its designed alignment for over 50 years. Under various Congressional authorities and the Missouri River BSNP, USACE performs annual maintenance, as well as emergency repairs, on bank stabilization and navigation structures along the channel of the Missouri River. If, during a flooding event, the channel migrates toward or away from the designed alignment, it is possible that USACE would move the channel back to its original alignment, especially in the vicinity of major infrastructure like CNS. However, depending on the severity of the flood damage, it may be some time before those repairs/channel realignments could be performed.

2.8.7 References

2.8-1. United States Nuclear Regulatory Commission (NRC). 2011. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America. NUREG/CR-7046, PNNL-20091. U.S. Department of Energy, Office of Nuclear Regulatory Research: Richland, Washington.

2.8-2. National Research Council. 2002. The Missouri River Ecosystem: Exploring the Prospects for Recovery Committee on Missouri River Ecosystem Science. National Academy Press, Washington, D.C.

2.8-3. U.S. Army Corps of Engineers (USACE). Missouri River Recovery Program. Accessed March 15, 2014. http://moriverrecovery.usace.army.mil/mrrpgis/.

2.8-4. U.S. Army Corps of Engineers (USACE). March 2006. Missouri River Mainstem Reservoir System, Master Water Control Manual, Missouri River Basin. USACE Northwest Division, Missouri River Basin, Omaha, NE.

2.8-5. Nebraska Maps and More 2014. School of Natural Resources Map and Publication Store. Lincoln, NE. http://nebraskamaps.unl.edu/productcart/pc/home.asp.

2.8-6. United States Geological Survey (USGS). 2013. USGS National Water Information System (NWIS). Washington, D.C. Accessed March 5, 2014. http://waterdata.usgs.gov/nwis/.

2.8-7. Nebraska Department of Natural Resources. NAIP aerial photography. Accessed March 15, 2014. http://dnr.nebraska.gov/data.

2.8-8. United States Nuclear Regulatory Commission (NRC). March 2007. Standard Review Plan. NUREG-0800, Section 2.4.9, Channel Diversions. U.S. Department of Energy, Office of Nuclear Regulatory Research: Richland, Washington.

2.8-9. U.S. Army Corps of Engineers (USACE). October 31, 1994. EM 1110-2-1418, Engineering and Design Channel Stability Assessment for Flood Control Projects. USACE, CECW-EH-D, Washington, DC 20314-1000. http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1418.pdf.

2.8-10. Lagasse, P.F., L.W. Zevenbergen, W.J. Spitz, and C.R. Thorne. August 2004. Methodology for Predicting Channel Migration. National Cooperative Highway Research Program (NCHRP) Web-Only Document 67 (Project 24-16). Prepared for National Cooperative Highway Research Program, Transportation Research Board. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w67.pdf.

2.8-11. U.S. Army Corps of Engineers (USACE). August 2012. Missouri River Stage Trends Technical Report. USACE Northwest Division, Missouri River Basin Water Management Division, Omaha, NE. http://www.nwd-mr.usace.army.mil/rcc/reports/pdfs/MRStageTrends2012.pdf.

2.8-12. United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS). December 1996. National Engineering Field Handbook, Part 650, Chapter 16. Streambank and Shoreline Protection Code 580, Companion Document 580-10.

2.8-13. United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS). 2013. Gridded Soil Survey Geographic (gSSURGO) Database for Nebraska. Accessed March 15, 2014. http://soils.usda.gov/survey/geography

2.8-14. U.S. Army Corps of Engineers (USACE). July 2011. Missouri River Navigation Charts - Sioux City, Iowa to Rulo, Nebraska. U.S. Army Engineer District, Omaha Edward Zorinsky Federal Building 1616 Capitol Ave. Omaha, Nebraska 68102.

2.8-15. U.S. Army Corps of Engineers (USACE). No date. After Action Report: Missouri River and Tributaries Flood of 2011. Received from John Remus, USACE, via email on October 1, 2013. See Attachment 5.

2.8-16. United States Geological Survey (USGS). 2014. Geomorphic Changes Caused by the 2011 Flood at Selected Sites Along the Lower Missouri River and Comparison to Historical Floods. U.S. Geological Survey Professional Paper 1798–H. http://dx.doi.org/10.3133/pp1798H.

2.8-17. United States Geological Survey (USGS). 2011. Earth Resources Observation & Science Center (EROS), accessed April 15, 2014. http://earthexplorer.usgs.gov/

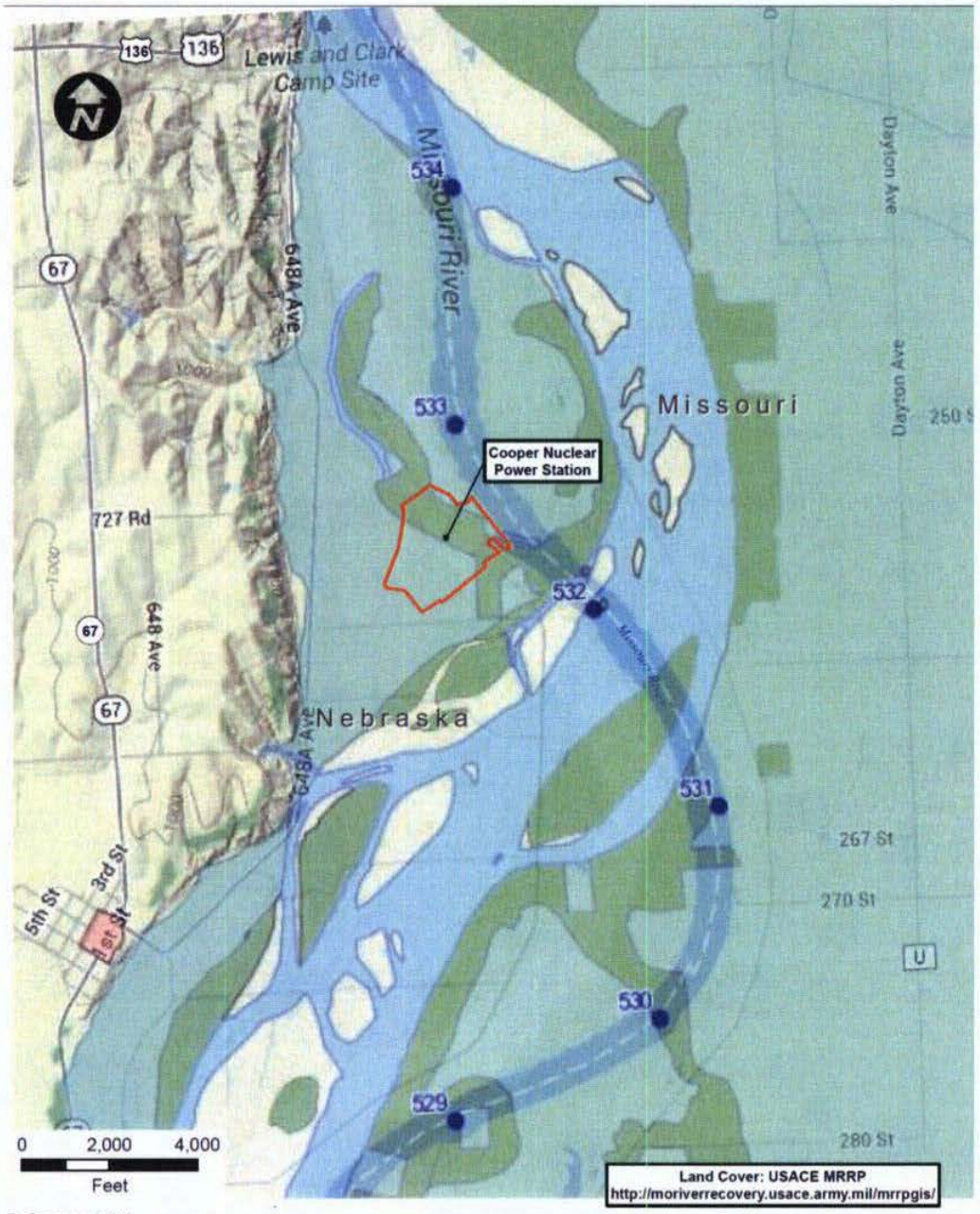
2.8-18. U.S. Army Corps of Engineers (USACE). CRREL Database. Accessed April 1, 2014. http://icejams.crrel.usace.army.mil/.

2.8-19. U.S. Army Corps of Engineers (USACE). March 2003. Final Supplemental Environmental Impact Statement for the Missouri River Fish and Wildlife Mitigation Project. USACE, Kansas City and Omaha Districts.

2.8.8 Figures

The figures associated with Section 2.8 are presented on the following pages.

Figure 2.8-1: 1879 Land Cover

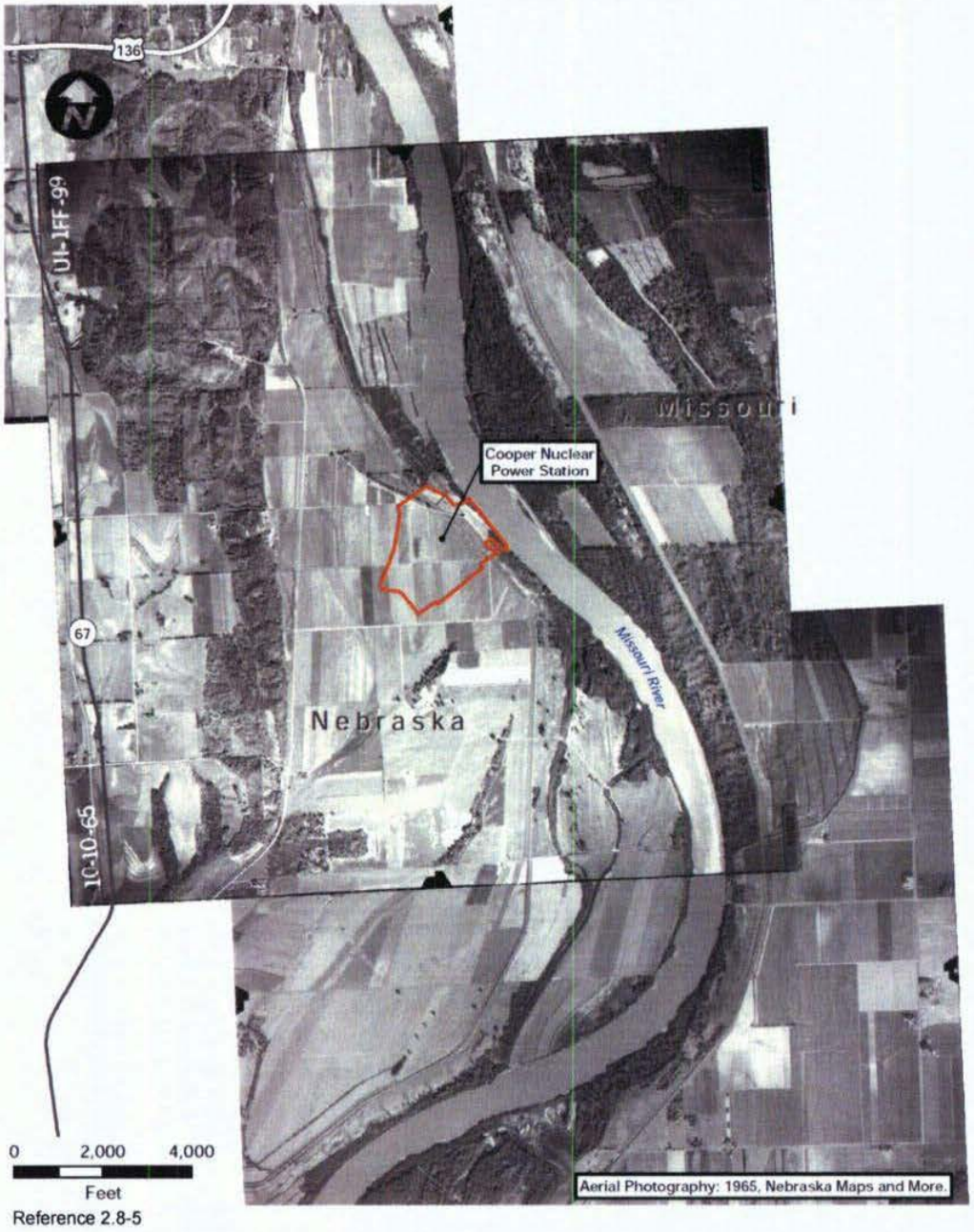


Reference 2.8-3

Figure 2.8-2: 1940 Aerial Photography

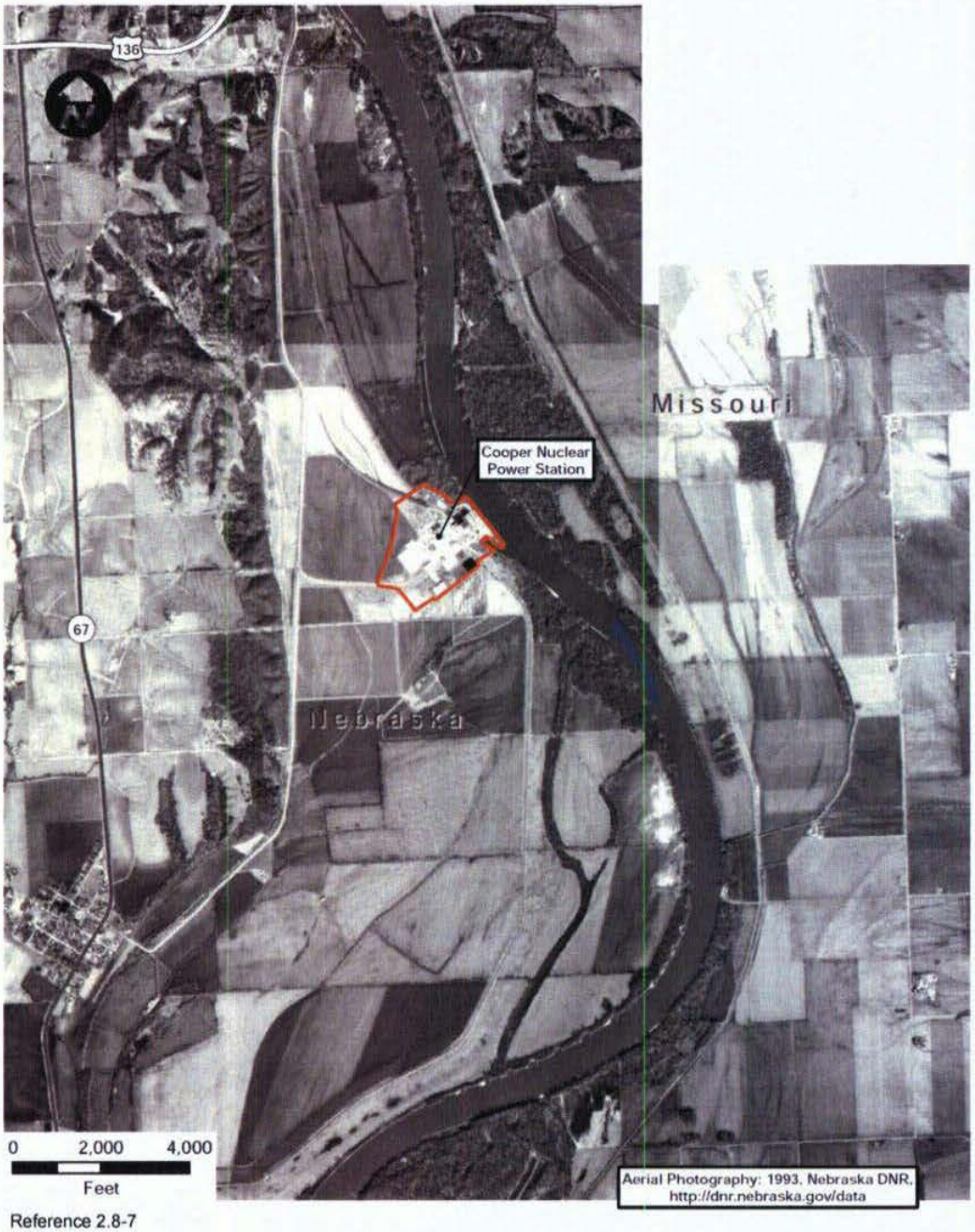
Reference 2.8-5

Figure 2.8-3: 1965 Aerial Photography



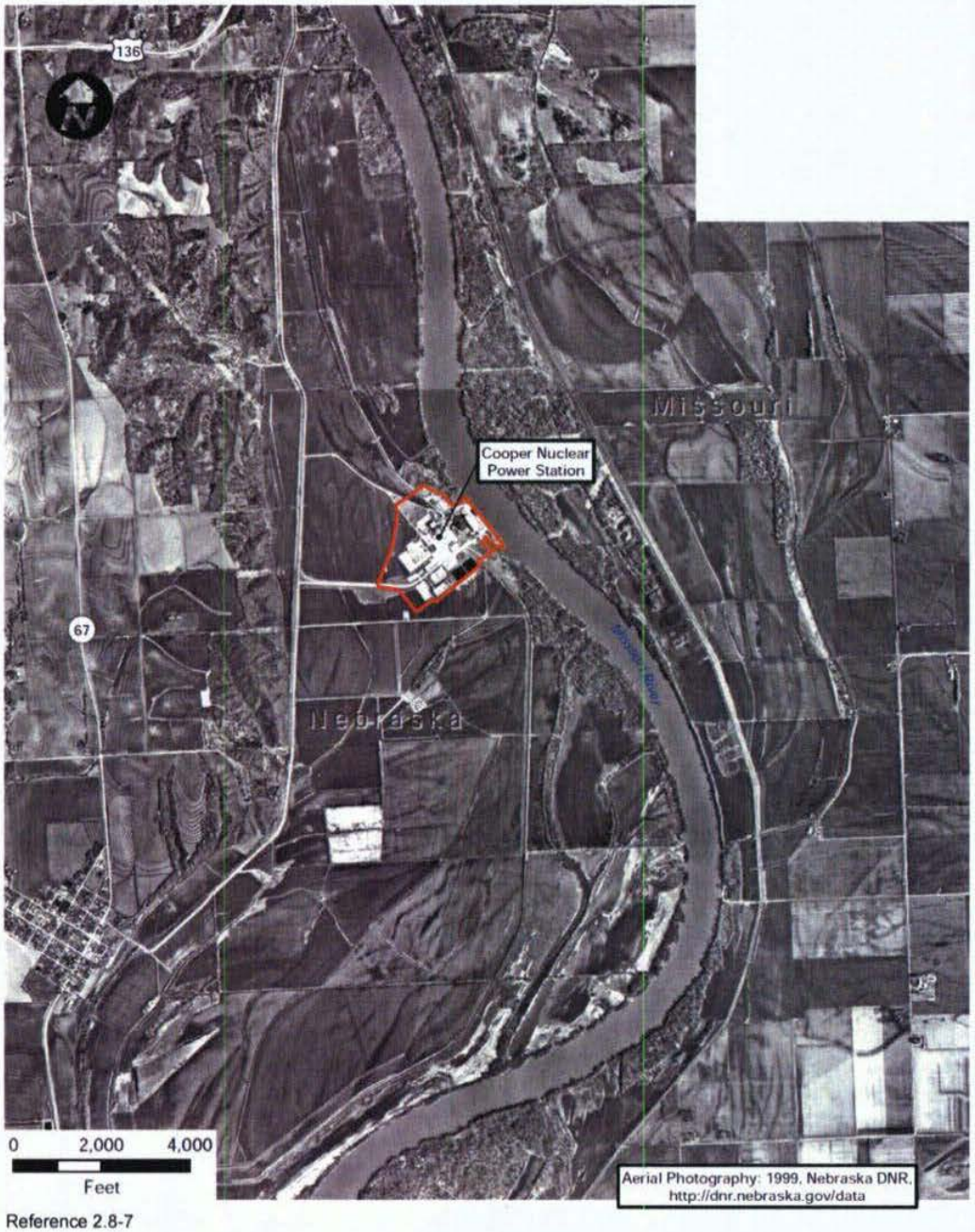
Reference 2.8-5

Figure 2.8-4: 1993 Aerial Photography



Reference 2.8-7

Figure 2.8-5: 1999 Aerial Photography



Reference 2.8-7

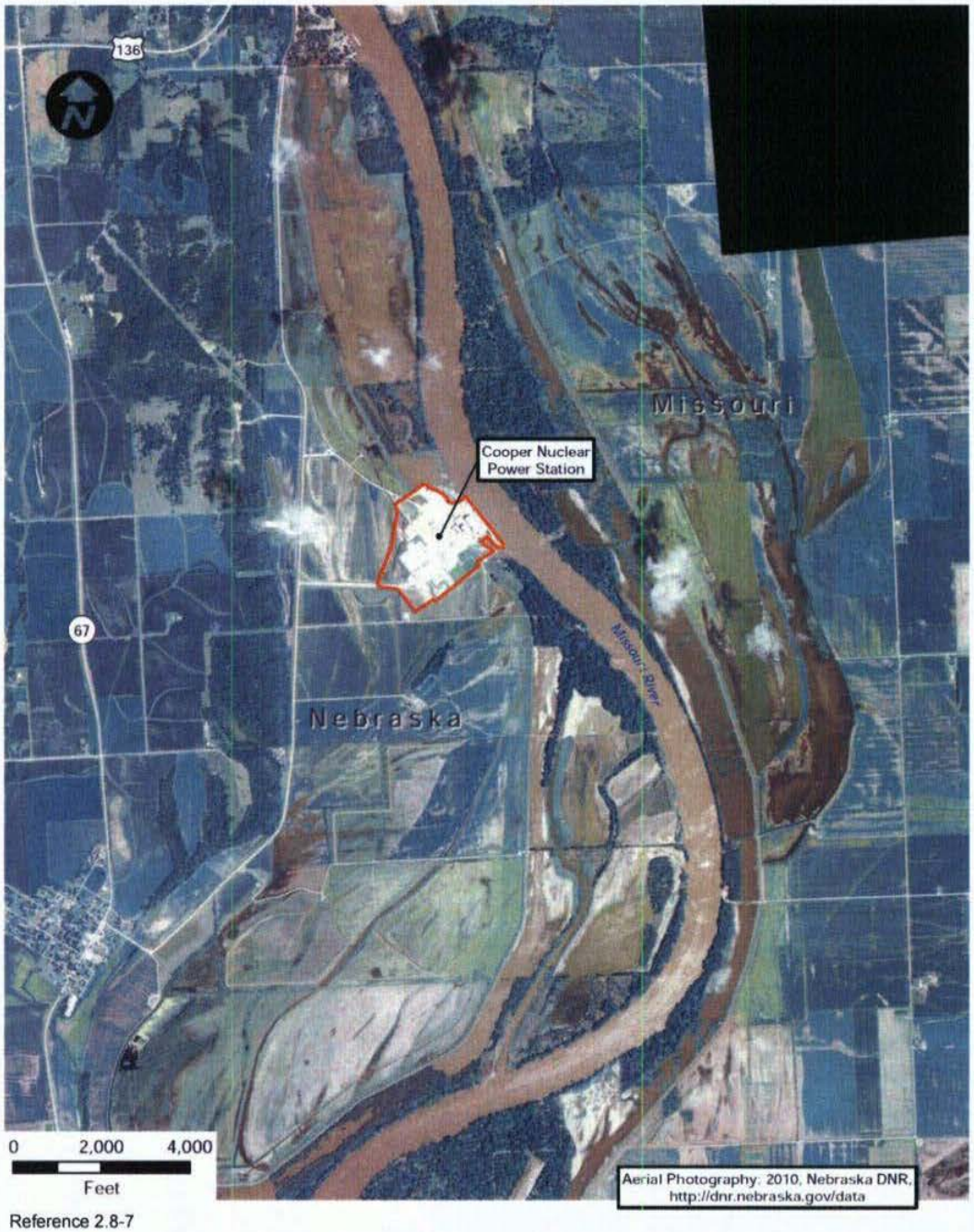
Figure 2.8-6: 2003 Aerial Photography

Reference 2.8-7

Figure 2.8-7: 2005 Aerial Photography

Reference 2.8-7

Figure 2.8-8: 2010 Aerial Photography

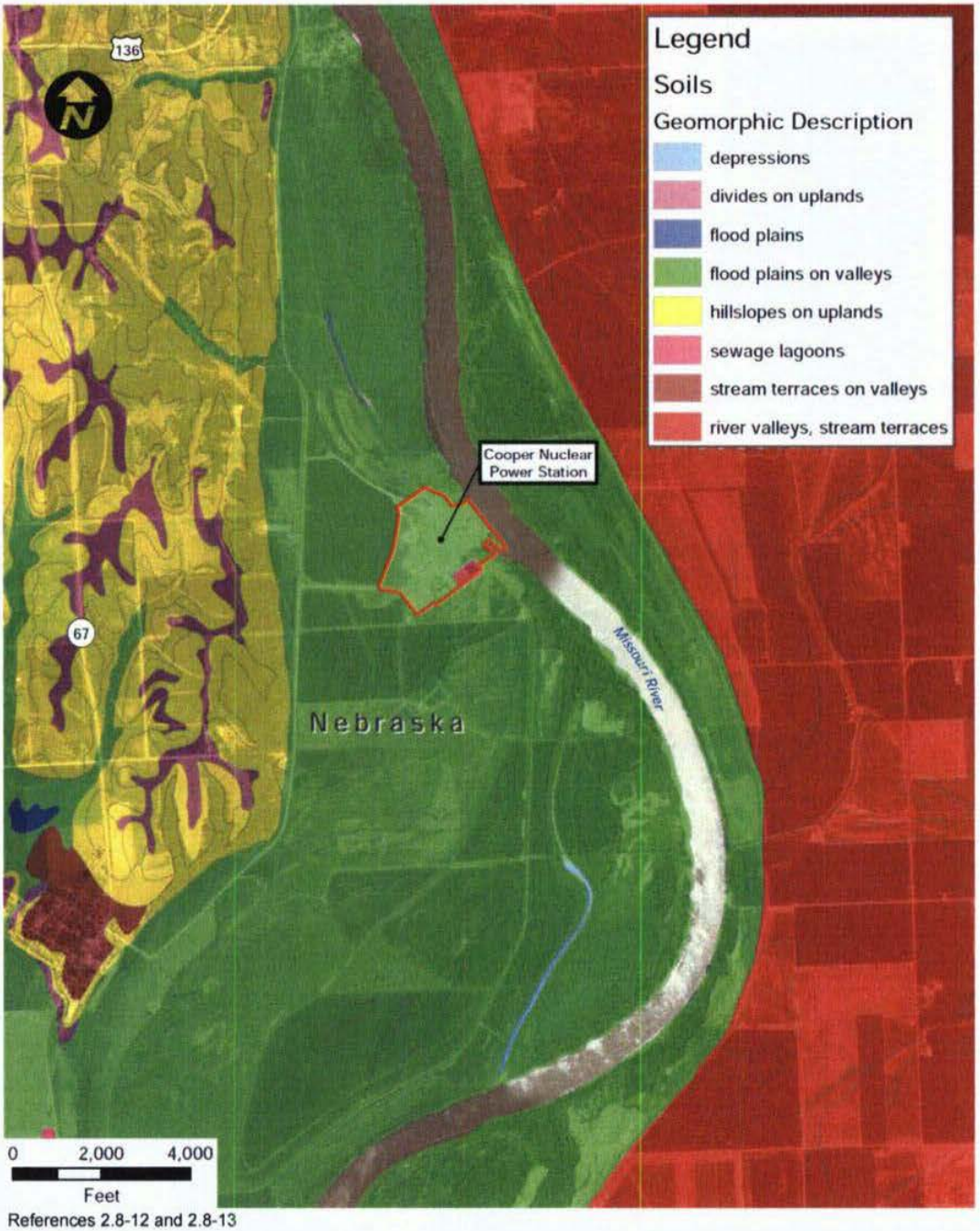


Reference 2.8-7

Figure 2.8-9: Hydric Soils

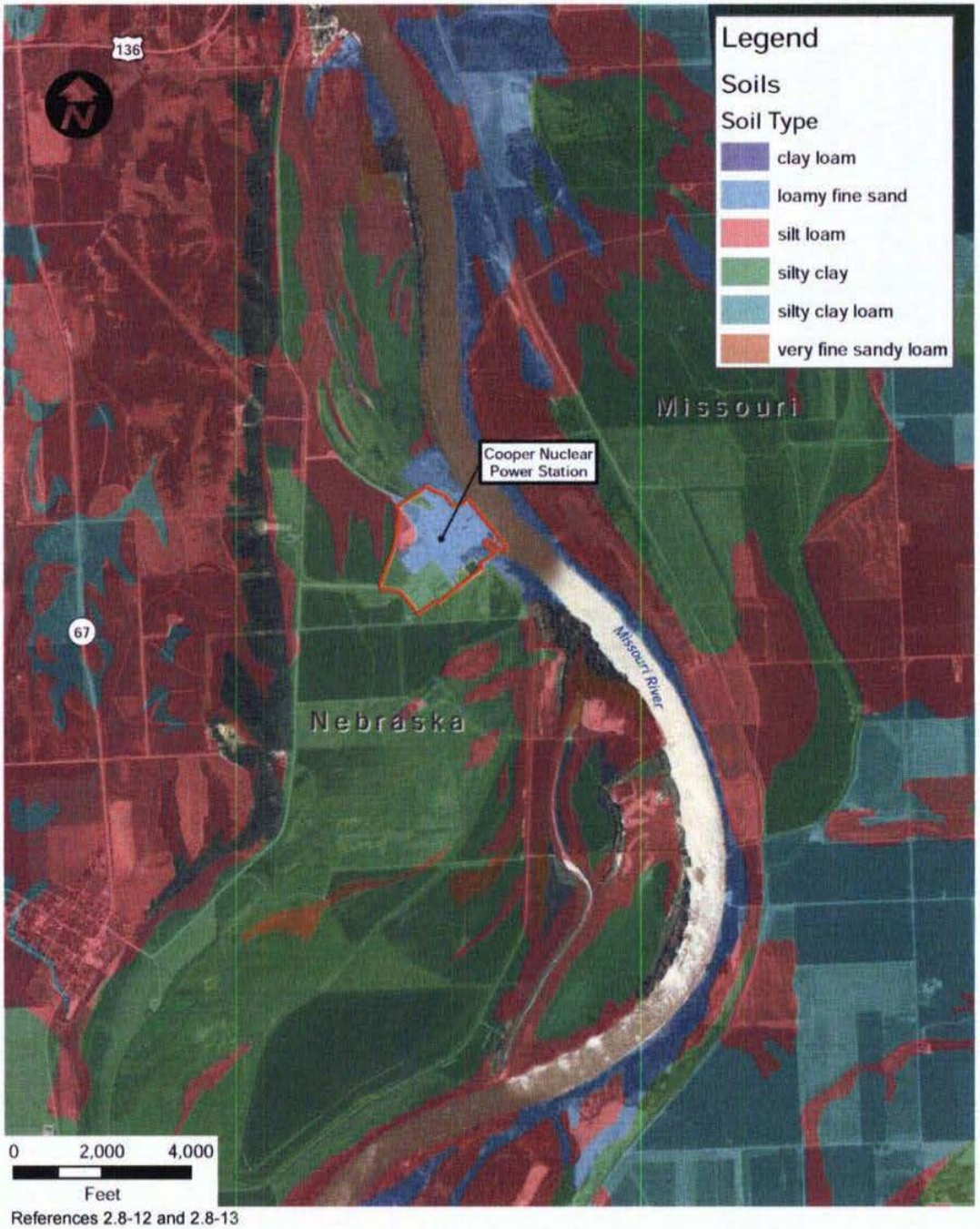
References 2.8-12 and 2.8-13

Figure 2.8-10: Geomorphic Soil Description



References 2.8-12 and 2.8-13

Figure 2.8-11: Soil Textures



References 2.8-12 and 2.8-13

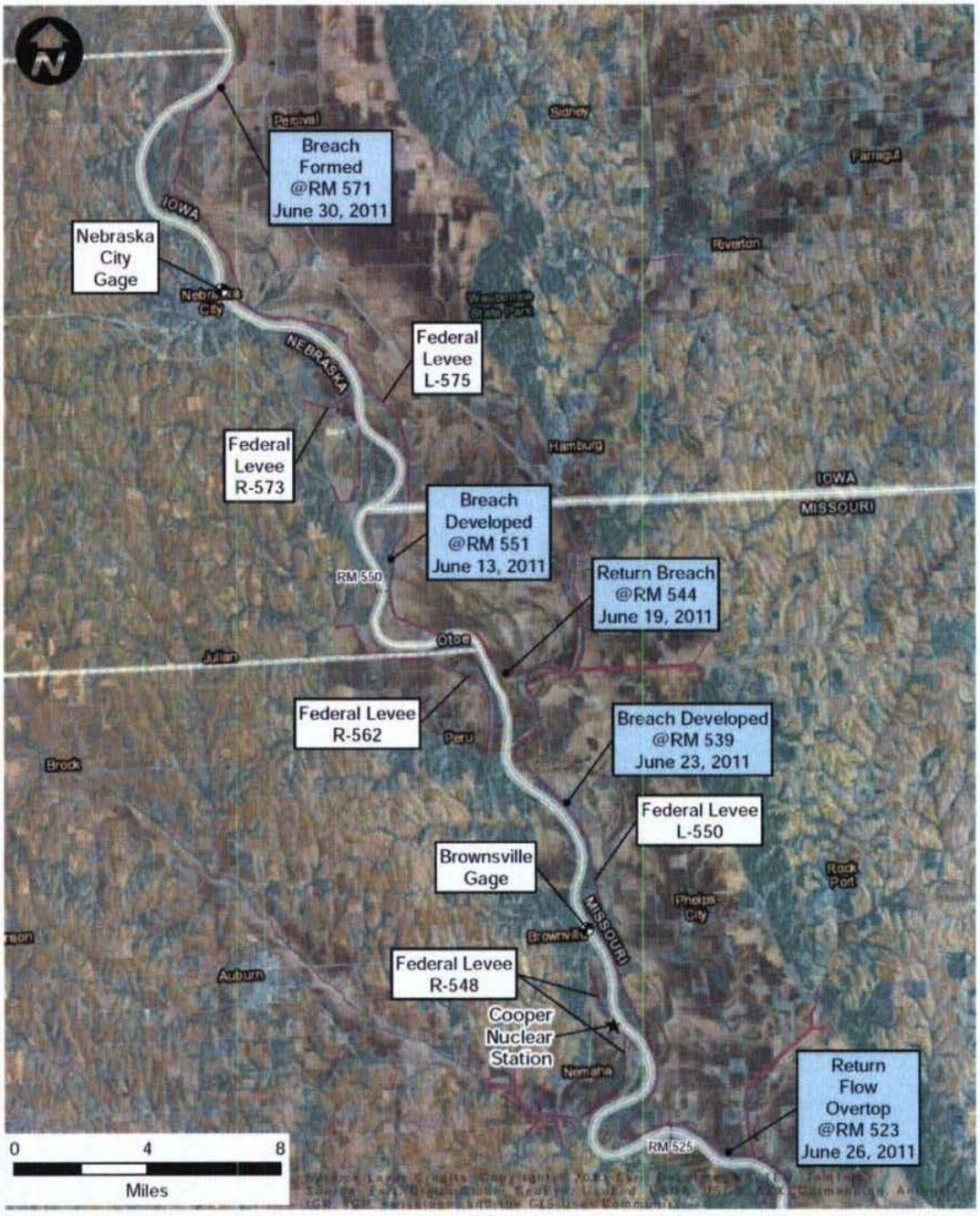
Figure 2.8-12: 2011 Landsat Thematic Mapper Surface Reflectance On-demand

Reference 2.8-17

Figure 2.8-13: 2012 Aerial Photography

Reference 2.8-7

Figure 2.8-14: 2011 Flood Breach Locations



2.9 COMBINED EFFECTS

The combined effect of different flood-causing mechanisms is discussed in Sections 2.1 through 2.8, where applicable. The combined effect flooding criteria for this reevaluation are based on the guidelines presented in ANSI/ANS-2.8-1992 (Reference 2.9-1) and NUREG/CR-7046 (Reference 2.9-2).

2.9.1 References

2.9-1. American Nuclear Society. 1992. ANSI/ANS-2.8-1992: Determining Design Basis Flooding at Power Reactor Sites. American Nuclear Society Publishing, La Grange Park, IL.

2.9-2. Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

3. COMPARISON OF CURRENT AND REEVALUATED FLOOD-CAUSING MECHANISMS

Table 3.14-1 summarizes the comparison of current and reevaluated maximum flood levels for all possible flood-causing mechanisms. The table indicates which of the reevaluated flood-causing mechanisms are bounded by the current licensing basis (CLB) flood hazard. The comparison shows that the reevaluated flood elevation due to dam failure event exceeds the current design basis flood elevation at the site. Additionally, the reevaluated hazard includes the associated flood effects of debris loading and flood duration, which were not considered in the CLB. The interim actions are identified in Section 4 of this report.

3.1 LOCAL INTENSE PRECIPITATION

The Cooper Nuclear Station (CNS) Updated Safety Analysis Report (USAR [Reference 3-1]) indicates that the site drainage is designed such that any excess rainfall not immediately absorbed into the ground will flow away from the buildings to be discharged into drywells or low-lying areas adjacent to the plant site. The plant area has a general site grade elevation of approximately 903 feet (ft) mean sea level (MSL [903.37 ft NAVD88]) and a finished floor elevation of 903.5 ft MSL (903.87 ft North American Vertical Datum of 1988 [NAVD88]).

The reevaluated flooding impacts due to the local intense precipitation (LIP) are presented in Section 2.1. The reevaluation used a combination of Hydrometeorological Reports 51 and 52 (HMR 51 and HMR 52) to obtain the point rainfall intensities and distributions. The rational method with conservative runoff coefficients (i.e.,1.0) was used for computation of peak probable maximum precipitation (PMP) runoff from different drainage areas of the plant site. The Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.1 computer model was used to determine the maximum water surface elevation (WSEL) and flow velocity. The analysis was performed assuming underground storm drains and culverts, as well as roof drains, are clogged and not functioning during the LIP event.

The simulated peak WSELs at the site varied between 903.56 ft NAVD88 and 903.87 ft NAVD88. The simulated peak flow velocities varied between 2.87 feet per second (fps) and 5.02 fps, which are not expected to produce any erosion hazards, as described in Section 2.1. However, velocities on the steep slope at the periphery of the main plant area may cause local erosion of the sloped surface, but there would be no effect on safety-related facilities. Since the USAR does not specifically consider flooding from an LIP event, and the reevaluated LIP event shows WSELs below the finished floor level, significant inundation above the finished floor elevations is not expected. However, the potential for minor flooding at the entrances of these structures may exist during the peak of the event.

3.2 FLOODING IN STREAMS AND RIVERS

The CNS USAR (Reference 3-1) refers to flooding in streams and rivers, also called probable maximum flood (PMF), analysis performed by the U.S. Army Corp of Engineers (USACE) and the U.S. Atomic

Energy Commission (AEC) with a maximum still water level of 903.37 ft NAVD88. This WSEL, concurrent with wind effects, would result in a maximum water level of 909.57 ft NAVD88 outside of the Intake Structure and 905.37 ft NAVD88 on other exposed systems, structures, and components (SSCs). These water levels (excluding Intake Structure combined WSEL of 909.57) are below the site flood protection level of 906.37 ft NAVD88 (Reference 3-1). A higher water level outside of the Intake Structure will not affect safety-related structures and components (Reference 3-1). The peak PMF discharge is reported as 600,000 cubic feet per second (cfs) in the USAR.

The reevaluation of flooding in streams and rivers is presented in Section 2.2. The reevaluation used a combination of HMR 51 and HMR 52 to establish the PMP storm depth, spatial distribution, centering, and orientation. Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software was used to apply the PMP storm on the Basin upstream of CNS and downstream of Gavins Point Dam to generate runoff hydrographs. To estimate the PMF water levels, a multi-leveled modeling approach was adopted in this flood hazard reevaluation study. A 310-mile basin-scale one-dimensional (1-D) HEC-RAS unsteady model was used to predict the PMP hydrograph. The results from the 1-D basin-scale routing model were then used to perform a reach-scale two-dimensional (2-D) hydraulic model. The results from this 2-D reach-scale model were used to predict WSELs and velocities at CNS for the PMF.

The reevaluated analysis resulted in a peak WSEL (PMF plus wind setup plus wave runup) of 904.1 ft NAVD88 and 908.4 ft NAVD88 at the embankments surrounding the CNS Main Building Complex and on the vertical wall of the Intake Structure, respectively. Both water levels are lower than the associated flood level reported in the USAR. The reevaluated peak PMF discharge is 769,135 cfs, which is greater than values reported in the USAR. Since flood protection levels of 906.37 ft NAVD88 for the MBC and 919.57 ft NAVD88 for the Intake Structure are higher than the reevaluated PMF water levels, CNS would not adversely impacted from hazards associated with flooding in streams and rivers.

3.3 DAM BREACHES AND FAILURES

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

3.4 STORM SURGE

The CNS USAR (Reference 3-1) does not evaluate flooding due to storm surge. Reevaluation of flooding due to storm surge, as discussed in Section 2.4, concludes that there is no plausible mechanism whereby a storm surge could cause flooding at the CNS site. Therefore, CNS is not impacted from hazards associated with flooding due to storm surge and this event is thus bounded by the CLB.

3.5 SEICHE

The CNS USAR (Reference 3-1) does not evaluate flooding due to a seiche. Reevaluation of flooding due to a seiche, as discussed in Section 2.5, concludes that there is no plausible mechanism whereby a seiche could cause flooding at the CNS site. Therefore, CNS would not be adversely impacted from hazards associated with flooding due to a seiche and this event is thus bounded by the CLB.

3.6 TSUNAMI

The CNS USAR (Reference 3-1) does not evaluate flooding due to a tsunami. Reevaluation of flooding due to a tsunami, as discussed in Section 2.6, concludes that there is no plausible mechanism whereby a tsunami could cause flooding at the CNS site. Therefore, CNS would not be adversely impacted from hazards associated with flooding due to a tsunami and this event is thus bounded by the CLB.

3.7 ICE-INDUCED FLOODING

Flooding due to an ice jam is qualitatively discussed from a low water level standpoint in the CNS USAR (Reference 3-1). The USAR further states that flooding caused by ice blockage is possible only at river levels significantly lower than the PMF.

Reevaluation of flooding due to an upstream ice jam failure, as discussed in Section 2.7, indicates that the peak WSEL resulting from an upstream ice jam breach is approximately 896.80 ft NAVD88. The peak WSEL at CNS as a result of backwater caused by an ice jam downstream of the plant is estimated to be 896.86 ft NAVD88. These calculated flood levels, which are highly conservative, indicate that CNS would not be adversely impacted from hazards associated with flooding due to an upstream ice jam break or downstream ice jam blocking the river. Since the WSEL associated with this flooding phenomenon does not exceed site grade, this event is considered bounded by the CLB.

3.8 CHANNEL MIGRATION OR DIVERSION

The CNS USAR (Reference 3-1) does not evaluate flooding due to channel migration or diversion, indicating a major channel diversion is unlikely after channel stabilization performed by USACE. However, the USAR discusses this mechanism only with respect to channel migration away from the site and the potential loss of the heat sink.

Reevaluation of flooding due to channel migration or diversion, as discussed in Section 2.8, indicates that the potential exists for the Missouri River to temporarily rechannelize into an old meander or divert away from the current navigational channel during the PMF and/or the flooding due to upstream dam failures. However, if, during a flooding event, the channel migrates toward or away from the designed alignment, it is possible that USACE would move the channel back to its original alignment, especially in the vicinity of a major infrastructure like CNS. In such a scenario, depending on the severity of the flood damage, it may be an extended length of time before those repairs/channel realignments could be performed. Therefore, CNS can be impacted from hazards associated with flooding due to channel migration or diversion and this event is not bounded by the CLB.

3.9 COMBINED EFFECTS

Combined effects of different flood-causing mechanisms are discussed in Sections 3.1 through 3.8, where applicable. The combined effects of wind setup and wave runup were considered in the reevaluation of the PMF and dam break hazards. The total WSEL at critical locations around the plant structures, as discussed in Section 3.3, are not bounded by the CLB. Therefore, CNS would adversely be impacted from combined effects of the dam break reevaluated hazard.

3.10 ASSOCIATED EFFECTS

An assessment of associated effects is required as discussed in Reference 3-3. The phrase "associated effects" is defined in Reference 3-4 as the hydrostatic and hydrodynamic loading, erosion and sedimentation, and debris impact effects caused by a plausible flood event. Reevaluation of these associated effects for both PMF and dam break events is discussed in Sections 2.2 and 2.3, respectively, where applicable.

3.10.1 Hydrostatic and Hydrodynamic Loads

The CNS USAR (Reference 3-1) states that principal structures are designed for a hydrostatic load of WSEL up to the flood protection level of 906.37 ft NAVD88. The USAR does not address hydrodynamic loads.

The reevaluated flood analysis considered the hydrostatic forces on the vertical face of the Intake Structure up to PMF WSEL of 903.0 ft NAVD88 and up to dam break WSEL of [redacted] ft NAVD88 on the vertical faces of the Main Building Complex. The reevaluated flood analysis considered the hydrodynamic forces on the vertical face of the Intake Structure up to PMF Combined WSEL of 908.4 ft NAVD88 and up to dam break combined WSEL of [redacted] ft NAVD88 on the vertical faces of the Main Building Complex. In addition to hydrostatic and hydrodynamic forces, current-induced forces were

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

estimated as part of the reevaluation study. Table 2.2-13 and Table 2.3-16 provide additional detail regarding loads calculated during the flood hazard reevaluation for PMF and dam break flood events, respectively. The comparison between CLB and reevaluated values shows that hydrostatic forces are bounded by CLB only during a PMF event. Hydrodynamic forces were not compared since they are not provided as part of the CLB.

3.10.2 Debris Loads

The CNS USAR does not specifically address the debris loads; however, it does discuss runaway barge details.

Debris forces were calculated using both maximum channel and overland velocities for PMF and dam break flood events as part of the flood hazard reevaluation study. The range of debris forces is presented in Table 3.14-2. The maximum PMF and dam break debris loads are 79,239 kips 167,800 kips, respectively. Table 2.2-14 and Table 2.3-17 provide details of debris loads calculated during the flood hazard reevaluation for PMF and dam break flood events, respectively. Debris loads were not compared since they are not provided as part of the CLB.

3.10.3 Erosion and Sedimentation

The CLB for CNS (Reference 3-1) does not include evaluation of the effects of erosion and sedimentation on critical safety-related structures. As discussed in Sections 2.1.5, 2.2.5.2, and 2.3.5.1, the reevaluated hazard from erosion and sedimentation during the LIP, PMF, and dam break events is considered minimal and localized. Therefore, CNS would not be adversely impacted by erosion and sedimentation related to any reevaluated flooding hazard and the reevaluated hazard is considered bounded by the CLB.

3.11 OTHER PERTINENT FACTORS

3.11.1 Flood Duration

As discussed in Section 2.3, flood duration is another pertinent factor in the assessment of the reevaluated flood hazard. Reference 3-4 defines flood duration as the total time comprising the preparations for a flood event, the period of inundation, and the recession of flood waters from the site. Specific details are provided in Table 2.3-7 through Table 2.3-10.

3.11.2 Overtopping

As discussed in Section 2.2.5.1, overtopping discharges are calculated using USACE coastal engineering manual equations for overtopped embankments during the PMF event. An overtopping discharge of 45 gallons per minute per linear foot was estimated.

3.11.3 Inundation

Section 9.4 of the Dam Failure Interim Staff Guidance (ISG) (Reference 3-2) recommends developing inundation maps to provide assistance in identifying SSCs important to safety that may require protective and/or mitigation measures from flooding due to a dam breach. As discussed in Section 2.3.5.2, a combination of HEC-RAS 4.1 and ArcGIS 10.2 software was used to map inundation limits for the most critical dam failure event ([redacted] dam hydrologic). The inundation limits shown in Figure 2.3-23 indicate that the plant site as well as [redacted] would be flooded.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

3.12 CONCLUSIONS

The comparisons between the CLB WSELs and the reevaluated flood hazard WSELs concluded that the safety-related structures at CNS are protected from flood-causing mechanisms once the flood protection features are in place. However, as discussed above, the dam break event WSEL [redacted]

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

3.13 REFERENCES

3-1. Updated Safety Analysis Report (USAR), Cooper Nuclear Station.

3-2. U.S. Nuclear Regulatory Commission, "Interim Staff Guidance for Estimating Flooding Hazards due to Dam Failure," JLD-ISG-2013-01, July 29, 2013.

3-3. U.S. Nuclear Regulatory Commission, "Trigger Conditions for Performing an Integrated Assessment and Due Date for Response," ML12326A912, December 3, 2012.

3-4. U.S. Nuclear Regulatory Commission, "Interim Staff Guidance for Performing the Integrated Assessment for External Flooding," JLD-ISG-2012-05, November 30, 2012.

3.14 TABLES

Tables associated with Section 3 are presented on the following pages.

Table 3.14-1: Current Licensing Flood Elevations and Reevaluated Flood-Causing Mechanisms

Flooding Mechanism	Current Maximum Flood Level (ft NAVD88)		Reevaluated Maximum Flood Level (ft NAVD88)	Comparison [4]
Local Intense Precipitation	Not specifically addressed in the USAR		903.87	Not described in the USAR
Flooding in Streams and Rivers	Other SSCs	905.37 [1]	904.1 [2]	Bounded
	Intake Structure	909.57 [1]	908.40 [2]	
Dam Breaches and Failures	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)		(b)(3):16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F) [3] / [2]	Not Bounded (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)
Storm Surge	Not plausible		Not plausible	Not plausible
Seiche	Not plausible		Not plausible	Not plausible
Tsunami	Not plausible		Not plausible	Not plausible
Ice-Induced Flooding	Not specifically addressed in the USAR		896.86 [3]	Not described in the USAR
Channel Migration or Diversion	The USAR is silent on potential site flooding due to channel migration or diversion. However, the USAR discusses this mechanism only with respect to channel migration away from the site and the potential loss of heat sink.		The potential for channel diversion during a flood event due to PMF or upstream dam failures could affect safety-related SSCs.	Not Bounded
Combined Effects	Not specifically addressed in the USAR		See Section 3.9	Not described in the USAR
Associated Effects	Not specifically addressed in the USAR		See Section 3.10	Not described in the USAR

Notes:
1. The flood levels are a combination of still water levels and wave runup.
2. The flood levels are a combination of still water levels, wind setup, and wave runup.
3. The flood levels are still water levels only.
4. Flood event is considered bounded if the reevaluated effect is lower than the current licensing basis.

Table 3.14-2: Debris Impact Loads Summary

Flood Event	Location	Maximum	Minimum
PMF (kip)	Channel	79,239	15.54
	Overland	21,441	4.2
Dam Break (kip)	Channel	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	
	Overland		

4. INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

4.1 REGULATORY BACKGROUND

The U.S. Nuclear Regulatory Commission (NRC) 50.54(f) letter of March 12, 2012 provides that flood hazard reevaluations be performed using present-day regulatory guidance and methodologies applicable to new nuclear plant applications. For the sites where the reevaluated flood hazard exceeds the design basis, licensees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard through the hazard evaluation.

Because the existing plants were not designed using the methods and assumptions given, any issues identified from the reevaluated hazards should not be treated as Current Licensing Basis (CLB) deficiencies or require operability review. If the reevaluated flood hazard at a site is not bounded by the current design basis, licensees are requested to perform an Integrated Assessment per NRC direction.

4.2 EVALUATION OF THE IMPACT OF THE REEVALUATED FLOOD LEVELS ON STRUCTURES, SYSTEMS, AND COMPONENTS (SSCs)

Section 2 of this report presents the reevaluation of flood hazards at Cooper Nuclear Station (CNS) using present-day regulatory guidance and methodologies. Section 3 of this report summarizes the comparison of the CLB flood elevation to the reevaluated flood elevation for each of the following flood-causing mechanisms:

- Local Intense Precipitation (LIP)
- Flooding in Streams and Rivers
- Dam Breaches and Failures
- Storm Surge - Not a credible hazard
- Seiche - Not a credible hazard
- Tsunami - Not a credible hazard
- Ice-Induced Flooding
- Channel Migration or Diversion

As stated in Section 3, the site would be affected by two flood-causing mechanisms, Dam Breaches and Failures and Channel Migration or Diversion. Interim actions associated with each of these flood-causing mechanisms are described in following sections.

4.3 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED FOR CNS

4.3.1 Dam Failure

The interim mitigation and monitoring plan was based on the fact that the U.S. Army Corps of Engineers (USACE) Upper Missouri Reservoir System has a very large storage capacity and that it is both unlikely and predictable in advance that the reservoirs will reach the hydrologic or sunny-day reservoir levels over the course of a water year. In cooperation with the USACE, the licensee will analyze the projected reservoir conditions based on snowpack and predicted runoff to determine the projected worst-case scenario for the current water year. The hydraulic model of the Missouri River developed for this flood hazard reevaluation will be used to estimate the maximum water surface elevation (WSEL) at CNS for the projected reservoir conditions. This yearly hazard evaluation would then be what the plant flood protection and/or mitigation must be capable of meeting for the following year until the reservoir forecasts are revised. To be certain that the current conditions do not exceed the analyzed conditions, a monitoring plan was established (Reference 4-1).

Several yearly forecast conditions are published by USACE for their mainstem dams. These include the Lower Basic – a lower than average condition, Basic – an average condition, and Upper Basic – an above average condition. These forecasts are updated on a monthly basis, but offer a band of WSEL in which the dams will operate. For CNS, a yearly hazard evaluation of the Upper Basic forecast was performed by using modified USACE results for the Sunny-Day [redacted] dam break described in Section 2.3.2.4 of this report. Sunny-day dam failures at [redacted] Dam and [redacted] Dam were evaluated by USACE and found not to cause a [redacted] The modified hydrograph was routed downstream to CNS by the river model developed for use in this reevaluation to produce the maximum expected dam break flood for the year.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

4.3.1.1 Dam Monitoring Plan

A procedure implementing the Monitoring Plan as well as trigger points for plant protection was established as a part of commitments NLS2014052-01 and NLS2014052-02 from Reference 4-1. In the Monitoring Plan, the volume of the reservoir system in the lower four dams is reviewed on an established frequency to determine if triggers have been met and to evaluate the continued adequacy of the yearly hazard evaluation.

4.3.1.2 Trigger Points

The trigger points were established to maintain flood protection and mitigation capabilities as necessary for dam failure. Because of the spontaneous nature of a sunny-day dam failure, if at any time the reservoir system volume is large enough to surpass the design flood conditions in the event of a dam failure, a Beyond Design Basis flood could occur. This means that the time available to deploy the mitigation equipment is limited to the time elapsed from the dam failure until the WSEL exceeds the design basis at the site. It is also possible that a System dam could fail coincident with a flood on the Missouri River downstream of the System. As such, some of the trigger actions have an alternate flood trigger value in addition to a reservoir trigger values to ensure that the equipment is in place before the site is rendered inaccessible.

As for the other USACE dams, CNS is built 1 foot higher than the levees in the surrounding floodplain and is therefore less subject to flooding. Because of this, the lower three System dams do not cause site flooding on their own.

4.3.1.3 Mitigation Equipment

A Beyond Design Basis flooding plan was established to maintain the core and spent fuel pool temperature below 200 degrees Fahrenheit for an extended period of time. This plan uses cooling towers and fan coil units in conjunction with several heat exchangers positioned on Main Building Complex rooftops. Much of the necessary equipment is currently off site but the companies will provide the necessary equipment immediately upon request. This plan will be refined as a part of the Integrated Assessment.

4.3.2 Channel Migration or Diversion

The CNS site buildings will be assessed for adequacy of the foundation while subject to the dam failure floods presented in the previous sections.

4.4 REFERENCE

4-1. Letter from Nebraska Public Power District to U.S. Nuclear Regulatory Commission, NLS2014052, "Nebraska Public Power District's Flood Hazard Reevaluation Interim Actions for Cooper Nuclear Station; Cooper Nuclear Station, Docket No 50-298, DPR-46," June 2014.

5. ADDITIONAL ACTIONS

No additional actions are required.